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AMERICAN SOCIETY

OF

CIVIL ENGINEERS

December, 1896

TRANSACTIONS - - VOL. XXXVI

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CONTENTS.

PAPERS.

NO.		PAGE
783	The Astoria City Water-Works.	
	By ARTHUR L. ADAMS	1
	Discussion on Paper No. 783.	
	By RUDOLPH HERING	50
	Correspondence on Paper No. 783.	
	By EDMUND B. WESTON	51
	S. BENT RUSSELL	52
	JAMES H. HARLOW	52
	E. SHERMAN GOULD	54
	FRANKLIN RIFFLE	56
	DAVID C. HENNY	58
	ARTHUR L. ADAMS	62
784	The Construction of a Light Mountain Railroad in the Republic of Colombia.	
	By E. J. CHIBAS	65
	Correspondence on Paper No. 784.	
	By ROBERT A. CUMMINGS	86
	E. SHERMAN GOULD	86
	José R. VILLALON	87
	E. J. CHIBAS	89
785	The Condition of Steel in Bridge Pins.	
	By A. C. CUNNINGHAM	91
	Discussion on Paper No. 785.	
	By WILLIAM METCALF	100
	Correspondence on Paper No. 785.	
	By GEORGE T. DEFOREST	102
	GEORGE S. MORISON	102
	A. C. CUNNINGHAM	104
786	Improving the Entrance to a Bar Harbor by a Single Jetty.	
	By T. W. SYMONS	109
	Discussion on Paper No. 786.	
	By G. F. ALLARDT	125
	G. H. MENDELL	127
	Correspondence on Paper No. 786.	
	By F. V. ABBOT	132
	LEWIS M. HAUPT	133
	T. W. SYMONS	135
787	Some General Notes on Ocean Waves and Wave Force.	
	By THEODORE COOPER	139
	Discussion on Paper No. 787.	
	By CHARLES MACDONALD	159
	Correspondence on Paper No. 787.	
	By RUDOLPH HERING	159
	ROBERT FLETCHER	161
	THEODORE COOPER	169

Digitized by Google

NO.		PAGE
788	A Water Power and Compressed Air Transmission Plant for the North Star Mining Company, Grass Valley, Cal.	
	By ARTHUR DE WINT FOOTE	171
	Discussion on Paper No. 788.	
	By S. S. WHEELER	191
	WILLIAM DOBLE	191
	Correspondence on Paper No. 788.	
	By C. G. YOUNG	192
	A. D. FOOTE	193
789	Flow of Water in Wrought and Cast-Iron Pipes from 28 to 48 Ins. in Diameter.	
	By ISAAC W. SMITH	197
	Correspondence on Paper No. 789.	
	By ARTHUR L. ADAMS	212
	L. J. LE CONTE	212
	ISAAC W. SMITH	214
790	The New Water-Works of Havana, Cuba.	
	By E. SHERMAN GOULD	217
	Discussion on Paper No. 790.	
	By THOMAS CURTIS CLARKE	232
	EDWARD WEGMANN	232
	E. J. CHIBAS	233
	Correspondence on Paper No. 790.	
	By G. L. CHRISTIAN	234
	EMILE LOW	234
	L. J. LE CONTE	236
	E. SHERMAN GOULD	237
791	The Suspension of Solids in Flowing Water.	
	By ELON HUNTINGTON HOOKER	239
	Discussion on Paper No. 791.	
	By T. C. CLARKE	325
	L. L. BUCK	325
	FOSTER CROWELL	326
	Correspondence on Paper No. 791.	
	By LEON PARTIOT	326
	A. FLAMANT	327
	WILLIAM STARLING	328
	GEORGE Y. WISNER	336
	L. J. LE CONTE	338
	ELON HUNTINGTON HOOKER	340
792	The Reconstruction of Grand River Bridge.	
	By W. A. ROGERS	341
	Correspondence on Paper No. 792.	
	By ONWARD BATES	356
	EMILE LOW	356
793	Suspension Bridges.—A Study.	
	By GEORGE S. MORISON	359
	Discussion on Paper No. 793.	
	By T. C. CLARKE	417
	JOSEPH MAYER	420
	F. COLLINGWOOD	427
	THEODORE COOPER	428
	E. GYBBON SPILSBURY	435
	W. H. BREITHAUPT	437

Digitized by Google

NO.		PAGE
	Correspondence on Paper No. 793.	
	By GUSTAV LINDENTHAL	439
	W. HILDENBRAND	454
	C. W. RAYMOND	458
	G. BOUSCAREN	462
	FOSTER CROWELL	464
	J. E. GREINER	467
	GUSTAVE KAUFMAN	469
	J. C. MEEM	470
	T. KENNARD THOMSON	471
	GEORGE S. MORISON	471
794	Experiments on the Protection of Steel and Aluminum Exposed to Sea Water.	
	By A. H. SABIN	483
	Discussion on Paper No. 794.	
	By FRANCIS T. BOWLES	494
	WILLIAM BARCLAY PARSONS	495
	W. M. STILES	496
	M. R. SHERRERD	496
	R. D. UPHAM	496
	Correspondence on Paper No. 795.	
	By FOSTER CROWELL	498
	R. P. HOBSON	500
	L. M. HASTINGS	504
	A. H. SABIN	506
795	A Resurvey of the Williamsport Division of the Philadelphia and Reading Railroad. (Abstract.)	
	By GEORGE D. SNYDER	509
796	The Proper Profile for Resisting Wave Action. (Abstract.)	
	By ROBERT FLETCHER	514

MEMOIRS OF DECEASED MEMBERS.

ALEXANDER DALLAS BACHE, Hon. M. Am. Soc. C. E.	522
ÉMILE MALÉZIEUX, Hon. M. Am. Soc. C. E.	524
JOSEPH G. TOTTEN, Hon. M. Am. Soc. C. E.	525
SQUIRE WHIPPLE, Hon. M. Am. Soc. C. E.	527
WILLIAM MILNOR ROBERTS, Past-President Am. Soc. C. E.	531
JOB ABBOTT, M. Am. Soc. C. E.	538
WILLIAM ALBERT ALLEN, M. Am. Soc. C. E.	539
JAMES BARNES, M. Am. Soc. C. E.	540
HENRY ISAAC BLISS, M. Am. Soc. C. E.	541
HENRY D. BLUNDEN, M. Am. Soc. C. E.	542
ROBERT BRIGGS, M. Am. Soc. C. E.	542
ROBERT LINAH COBB, M. Am. Soc. C. E.	545
ZERAH COLBURN, M. Am. Soc. C. E.	546
ADDISON CONNOR, M. Am. Soc. C. E.	551
ECKLEY BRINTON COXE, M. Am. Soc. C. E.	552
HORACE LA FAYETTE EATON, M. Am. Soc. C. E.	554
JOHN ROBERTS GILLISS, M. Am. Soc. C. E.	555
WILLIAM HARRISON GRANT, M. Am. Soc. C. E.	557

Digitized by Google

	PAGE
Robert G. Hatfield, M. Am. Soc. C. E.	558
Norman James Nichols, M. Am. Soc. C. E.	559
Albert Franklin Noyes, M. Am. Soc. C. E.	560
Henry Ward Beecher Phinney, M. Am. Soc. C. E.	563
Thomas Prosser, M. Am. Soc. C. E.	564
Willard Smith Pope, M. Am. Soc. C. E.	565
James Clarence Post, M. Am. Soc. C. E.	569
Henry Frederick Rudloff, M. Am. Soc. C. E.	570
Isaac Munroe St. John, M. Am. Soc. C. E.	571
Howard Schuyler, M. Am. Soc. C. E.	572
Thomas Jennings Seely, M. Am. Soc. C. E.	574
Samuel Henry Shreve, M. Am. Soc. C. E.	576
Frederick Ellsworth Sickels, M. Am. Soc. C. E.	577
Wilmon W. C. Sites, M. Am. Soc. C. E.	582
John Chambers Thompson, M. Am. Soc. C. E.	584
Charles Truesdell, M. Am. Soc. C. E.	585
Louis Roberts Walton, M. Am. Soc. C. E.	586
Orlando Belina Wheeler, M. Am. Soc. C. E.	587
John Allston Wilson, M. Am. Soc. C. E.	588
James Hugh Stanwood, Assoc. M. Am. Soc. C. E.	590
Charles Wood, Assoc. M. Am. Soc. C. E.	591
Frank Berresford, Jun. Am. Soc. C. E.	594
William Alexis George Emonts, Jun. Am. Soc. C. E.	594
Vernon Hill Gridley, Jun. Am. Soc. C. E.	595
Russell Wadsworth Hildreth, Jun. Am. Soc. C. E.	596
Albert Jacob Stahlberg, Jun. Am. Soc. C. E.	597
William Howland Aspinwall, F. Am. Soc. C. E.	598
George Washington Cass, Jr., F. Am. Soc. C. E.	599
Thomas C. Durant, F. Am. Soc. C. E.	602
Sidney Dillon, F. Am. Soc. C. E.	603
Henry Farnam, F. Am. Soc. C. E.	605
Alfred Krupp, F. Am. Soc. C. E.	609
William C. Kingsley, F. Am. Soc. C. E.	612
Frederick W. Merz, F. Am. Soc. C. E.	615
McRee Swift, F. Am. Soc. C. E.	616

ILLUSTRATIONS.

PLATE.		PAPER.	PAGE
I.	Fig. 1. Apparatus for Measuring Tension in Stave Bands	783	19
	" 2. View of Astoria Reservoir during Construction	783	19
II.	Views on Line of Mountain Railroad	784	71
III.	" " " "	784	83
IV.	Chart of Grays Harbor, Wash.	786	111
V.	Views of Aqueduct at Grass Valley, Cal.	788	175
VI.	Water-Works and Air Compressors, Grass Valley, Cal.	788	179
VII.	Views of Havana Reservoir under Construction.	790	223
VIII.	" " " Completed	790	229
IX.	Views of Grand River Bridge	792	358
	Views of Metal Plates Exposed to Action of Fresh Water	794	505

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No. 783.

THE ASTORIA CITY WATER-WORKS.

By ARTHUR L. ADAMS, M. Am. Soc. C. E.

PRESENTED MAY 6TH, 1896.

WITH DISCUSSION.

It is an attractive feature of the engineer's vocation that each engagement presents for solution new and often interesting problems which tax both skill and ingenuity, and it is seldom that the results, when carefully observed and properly recorded, are not of both interest and profit to others in the profession. The author proposes to make brief mention of the old water-works of Astoria, Ore., and to present a description of the works just completed, and to accompany it with such notes and observations incident to construction, cost and final determination of results as many engineers in charge of work record in their private note-books, but, at the expense of the profession at large, often fail to make a matter of public recital. In the amount of expenditure involved these works are in no way exceptional, but in variety of work and consequent interest, it is believed that they have not been often exceeded, even by works of considerably greater magnitude.

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The Old Works.

The city of Astoria, first established in 1811 by John Jacob Astor, and made memorable by Washington Irving's "Astoria," is situated on the south shore of the Columbia River, here 7 miles in width, about 12 miles above its mouth, and occupies a much broken peninsula rising to a height of 600 ft. between the Columbia River and Young's Bay.

In 1883–84, the Columbia Water Company, a private corporation, built a system of works utilizing Bear Creek, a small mountain stream flowing into the Columbia about 7 miles above the town, as a source of supply. The water was conducted by gravity to the town through a line of lap-welded wrought-iron pipe of the following sizes and lengths: 1 825 ft. of 10-in., 2 275 ft. of 8-in., 59 000 ft. of 6-in. The line terminated in a reservoir of 500 000 galls. capacity built at a flow-line elevation above mean low tide of 166 ft., from which the water was distributed over the town through pipes ranging in size from 6 ins. to 1 in., the smaller sizes largely predominating. No provision was made for fire protection from these works. No especial interest attaches to them other than as showing a remarkable example of how works may be built by so-called practical men without incurring any expense for engineering supervision; 140 000 galls. in 24 hours is all that the pipe-line could ever be made to deliver into the reservoir, not over 60 or 70% of what might reasonably have been expected if properly built. The line was laid from the source immediately to tide water in the Columbia, without recourse to instrumental work, and thence down the tide flats to town, thus subjecting the entire line to the greatest pressure possible, and to the destructive action of the salt water, which in the course of a very few years so thoroughly honeycombed the pipe as to render it a very serious question from day to day whether or no water could be supplied to consumers on the morrow. In addition to this the pipe was laid without regard to either alignment or grade, and with so little cover that the lateral components of the thrust at sharp angles frequently pulled the joints entirely apart.

This train of evils consequent upon stupid work led in 1891 to an agitation on the part of the citizens which culminated in the appointment by an act of the State Legislature of a Board of Water Commissioners, authorized to purchase the works of the Columbia Water Company, to reconstruct the same, or to build new works with a view

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to greater efficiency and supply, and to issue municipal bonds in an amount not exceeding $500 000 necessary for the accomplishment of these ends.

Shortly after its organization, the Commission purchased the old works for the sum of $75 000. Some effort was then made to secure an improvement of the service without resorting at once to entire reconstruction, but without any satisfactory results being attained, while repeated interruptions in the water supply by failure in the gravity line, the insufficiency of the supply, and the inability of the system as built to afford water to any of the higher elevations of the city, all rendered apparent the necessity for speedy construction of new works.

The New Works.

In November of 1893 the author reported for the Commission on an increased water supply, and recommended the construction of a new system substantially as has been subsequently carried out. During the following spring a beginning was made on the preliminary surveys and plans, but the general business depression of 1893 influenced the Commission to suspend operations indefinitely. In July, 1894, however, matters were again taken up with the intention of getting construction under way the following spring. In order to meet in a measure the pressing public demand for more water pending the construction of the new works, the old gravity line was parted at tide-water elevation, about 8 000 ft. distant from the reservoir, and diverted into a tank, thereby securing about 160 ft. more fall in the pipe line and an increase of one-half in the discharge. From the tank the water was pumped to the reservoir by arrangement with the electric street railway company to supply the power. The plans for the new works were completed and the programme for construction carried out as previously purposed.

Water Supply.—The water supply is derived from Bear Creek, the diversion being made about 1 mile farther up the stream than was selected in the construction of the old works. This is a beautiful mountain stream having a drainage area above the diverting point of 4.82 square miles, all of which is heavily timbered and covered with a dense growth of moss and ferns. During the rainy season the run-off varies from 10 000 000 to 30 000 000 galls. in 24 hours, and the dense vegetation serves to retain and prolong the supply during the few

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weeks of dry weather in August and September to an extent very unusual in a stream having so limited a drainage area. The yield of this area was materially increased by diverting into Bear Creek above the head works another stream, Cedar Creek, which originally entered a mile farther down, this work being accomplished at almost no expense. Careful weir measurements of the flow of the stream during the lowest stages show the following results:

1890.....	5 118 000	galls. in	24 hours	below mouth of Cedar Creek.
1891.....	4 763 000	"	"	" "
1892.....	4 280 000	"	"	" "
1893.....	2 646 000	"	above	" "
1894.....	2 400 000	"	"	" "
1893.....	718 000	"	in Cedar Creek.	

From these results and because of the ease with which about 50 000 000 galls. can be stored just above the diverting weir, it was assumed that a daily supply of 4 000 000 galls. could be made available from this source with a reasonable degree of certainty. It was accordingly determined that this figure should be made the basis for all computations of capacity in the new works. The construction of the storage reservoir was, however, to be deferred until the demand for a supply in excess of that afforded by the stream direct should necessitate its construction. This determination as to capacity in the construction of the new works was also influenced by the rapid growth of the town, now having a population of fully 10 000, and by the fact that the large and available sources of supply for the future city will all be brought in over the same general route adopted for the new line.

General Arrangement.—Briefly outlined, the new works consist of the following structures: The head works on Bear Creek, a small masonry diverting weir with a crest elevation of 589.35 ft., diverts the water through a head gate into a small receiving basin, from which it is led through an 18-in. wood-stave pipe to a masonry settling basin 1 000 ft. distant, where it is screened and weired. The water is then conducted through pipes of the following character and order: 11 956 ft. of 18-in. wood-stave pipe, 1 239 ft. of 16-in. riveted steel pipe, 10 450 ft. of 18-in. wood pipe, 2 413 ft. of 16-in. steel pipe, 3 606 ft. of 18-in. wood pipe, 12 776 ft. of 16-in. steel pipe, and 13 082 ft. of 18-in.

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wood pipe—in all a little over 10.5 miles, to an elevation within the city limits of 425.75 ft. Thence it passes by a rapid descent through 5 574 ft. of 14-in. steel riveted pipe to the power and gate house at the new reservoir. The elevation of the water surface of the reservoir when full is 282.4 ft., and of the point of power development 289.00 ft. The reservoir has a capacity of 6 250 000 galls. and is located on the Young's Bay side of the peninsula, while the city is situated principally on the Columbia side. From the reservoir the water is conveyed to the distributing system through an 18-in. pipe laid in a tunnel 1 200 ft. long, passing through the divide. The distribution is divided at present into a low and high service, the lower being supplied from the small reservoir previously connected with the old works, and the high service from the new reservoir. These two services, in case of fire, are thrown together, the lower reservoir cut out by the operation of a check valve, and the pressure secured from the upper reservoir. This end is accomplished by the operation of two hydraulic lift gates which are both opened and closed from the central fire station by means of a special hydraulic gate governor designed for this purpose. Two other services of greater elevation will eventually be added, three of the four being supplied by gravity. The fourth will be supplied by pumping with water power at the new power house.

Construction.—It was expected that by letting all contracts for materials and construction during the winter months, the contractors would be able to complete the work during the dry season, usually lasting in this locality from June 1st to October 1st. Accordingly advertisement was made in December, and bids opened on January 10th, 1895. Proposals were invited on the work divided into seven divisions, each being segregated into the different items entering into it, and a percentage of reduction from the aggregate amount of the proposal asked in consideration of the entire seven divisions being awarded to the same person. The divisions referred to were as follows:

First.—Clearing and grubbing the conduit right of way and grading the road alongside it; making necessary bridges and culverts; constructing a telephone line; excavating and refilling the conduit trench, and excavating the reservoir.

Second.—Head works, diverting weir and settling basin.

Third.—Lining reservoir, erecting gate and power house, including the furnishing and placing of all gates, fittings and appliances in it.

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Fourth.—Wooden stave conduit, furnishing and laying.

Fifth.—Riveted steel conduit, furnishing and laying.

Sixth.—Distributing system.

Seventh.—Tunnel.

Eighth.—All cement required on the work.

The almost entire absence of construction work on the Pacific Coast at this time rendered the bidding exceedingly spirited, with the final result that the Commission was confronted with the alternative of awarding several important divisions of the work on proposals, which, though formal in the last degree, and supported by contractors determined to have their rights recognized and to secure the work at any cost, were manifestly less in amount than that for which the work could be performed; or on the other hand, rejecting these bids and awarding to higher bidders. The author holds, in the much-discussed question to which this gives rise, that though a private person or company may often, with creditable discretion, discard a bid that is too low, yet in handling the funds of a municipality, if the proposal is strictly formal and the sureties satisfactory, the administrator of such funds cannot with due propriety and proper regard for the wishes of the public whom it serves discard such a bid. The subject has proved an interesting one in Astoria in the light of after developments. After a careful consideration of the matter, the awards were made to the lowest bidder in each case, and divisions 1, 2, 3, 6 and 7 were let in accordance with the above hypothesis, all but No. 6 being united in one award. Nos. 5, 6 and 8 were each awarded separately to different parties. While it was to be expected that some unpleasant conflict of interest would arise from this dividing of the work, yet the large saving to the city thereby was considered amply compensative.

Proposals were invited on $200 000 of bonds at the same time that bids were asked on the construction, but through certain unexpected difficulties in their negotiation, the money was not realized on them until the following May, when the contractors were immediately notified to proceed with the work. In the mean time the contracts for construction had been held in trust by a third party, to be delivered on the mutual consent of the principals.

The delay in getting the work started, and the necessity of ordering most of the steel used in the construction from the East at a time when the suddenly increased demand made it exceedingly difficult to get

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orders from a distance filled, caused it to be very late in the summer before work was well started on several of the contracts.

Affairs moved along as smoothly as can be expected on work where the majority of the contractors are losing money, until the middle of September, when the contractor for divisions 1, 2, 3 and 7 suddenly failed, and disappeared to escape the vengeance of some hundreds of unpaid laborers. The customary suits, attachments and injunctions immediately followed. A few thousand dollars allowed on estimates, still in the hands of the Commission to the credit of the bankrupt concern, neither public funds nor public works being attachable in Oregon, were by the order of the court paid over to a receiver. The Italian element, which largely predominated among the laborers, not being satisfied with the slow process of law in the recovery of their claims, and being incited by irresponsible and incendiary agitators, after trying for some days to force the payment of their employer's private debts out of the public funds in the hands of the Water Commission by parading the streets in full force in martial order and by besieging the offices and residences of the members of the Commission and of the engineer, finally inaugurated a strike against the city. Arming themselves with guns, axes and clubs they forced the suspension of all work outside the immediate confines of the town, notified all parties that any attempt to resume, until they had been paid in full, would be at the peril of their lives, and that unless they were speedily settled with, the works already built would be destroyed with giant powder. The Commission resisted the revilings and ill-considered advice of a large but misguided portion of the citizens, and steadfastly refused to incur a single expenditure not legally authorized. In the mean time arrangements were made with the bondsmen of the defunct company to proceed with the work, and public sentiment was satisfied when all strikers were offered work, payment of wages to be made weekly, and guaranteed by the city. As was anticipated, the strikers, misinterpreting these actions, attributed them to fear instead of sympathy, and remained as obdurate as ever, refusing to change their position until all claims for back wages were satisfied. This being impossible, however desirable, through lack of suitable legislation in Oregon, and public sympathy being then largely withdrawn, a determined effort was made to start the work, which was without much difficulty effected by the moral suasion of 25 Winches-

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ters. Most of the men returned to work in the course of a few days; but after trying in vain for two weeks to get decent and expeditious work from this element, which, through the influence of agitators and Italian lawyers, constantly maintained a menacing attitude, the author gave up in despair and had the entire force summarily removed from the work, and new laborers employed.

The gravity line was completed, and the water formally turned into the new reservoir on December 21st, in the presence of a large number of enthusiastic citizens. Although this was ostensibly the first water to enter any part of the new work, perhaps it is unnecessary to add that everything had been very quietly but thoroughly tested beforehand; and any imperfections in workmanship, liable to grow into a mountain of difficulty in the eyes of the populace, were well taken care of in advance.

The tunnel was completed on February 22d, and water was turned through from the reservoir into the low-service distribution about February 10th. In the meantime, however, the city had for some weeks been supplied from the new conduit by laying a temporary line of pipe up over the tunnel point and down to the low-service reservoir. The high service is not yet quite complete, as most of the pipe used in it was taken from the upper part of the old gravity line and from the old distribution, which could not be disturbed until the water supply was available from the new source and through the new distribution. The entire plant will have been completed in less than one year, although the larger part of the work has been done during the rainy season, not a light matter where the average annual rainfall is in excess of 75 ins. The prosecution of the work as a whole was very trying to both contractors and engineers. To the former, because only one succeeded in completing his work without sustaining a loss, to the latter because of the extreme difficulty always experienced in enforcing the provisions of a contract under such conditions. By a combination of circumstances, fortunate to the city, the works have been built at a remarkably small cost; and the author doubts if there is another system on the Pacific Coast where similar results have been accomplished with a smaller expenditure.

During construction, daily reports of force, materials used, and progress made on each part of the work, were made to the Chief Engineer; and these have been made the basis of carefully prepared esti-

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mates of cost, which are submitted in part with this paper. They may be considered entirely reliable. These reports, by the way, have proved invaluable in unraveling the affairs of the contracting company that failed, and in settling many claims regarding extras, etc.

Head Works.

The diverting weir is located near the head of a narrow, rocky and precipitous gorge on Bear Creek. It is very simple in character, is founded on solid rock, is entirely tight, and serves to divert the water through the head gate into a small receiving chamber, from which it enters the pipe leading to the settling basin, while the surplus water passes on over the weir. The masonry is of rough rubble basalt, quarried in the immediate vicinity, the face stones being squared. The mortar is made in the proportions of two to one of sand and imported Portland cement. The receiving chamber is covered with a frame and trap door securely bolted to the masonry.

The settling basin is located 1 000 ft. below the diverting weir. It consists of a masonry basin 50 ft. long by 8 ft. in width, having an average depth of 6 ft. The water enters at the end, flows the length of the basin, where, after passing through duplicate screens and over a measuring weir, it enters the pipe line, which continues uninterrupted to the city reservoir. The masonry is of the same general character as that of the diverting weir, and is finished on the inside with ½ in. of cement mortar, covered with two coats of asphaltum. The screens are of No. 12 sheet brass, perforated with ¼-in. holes, framed with two 1-in. angle irons, and rest in channel iron guides set in the masonry. They are handled by means of a block and tackle hung from a traveler. The measuring weir is of iron plate, has contractions and is provided with a weir gauge by which the gallons of water passing the weir in 24 hours are indicated on a graduated brass scale mounted on an iron pedestal. The index rises and falls with a copper float operating in a stand-pipe set in the wall of the basin, and communicating by means of a pipe with the water below the screens and above the weir. The entire settling basin is enclosed in a corrugated iron-covered structure. This basin reduces the velocity of flow to a rate that results in the precipitation of all heavy matter, and the screens stop all leaves, moss and fir needles with which the water is heavily charged during the rainy reason. Cleaning the screens, which

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is usually accomplished with a piece of square rubber packing fastened to the edge of a wooden hoe, is necessary about twice a day during the wet season in order to maintain a full flow in the pipe line. Provision is made for flushing out the basin from the bottom, and for discharging the surplus waters through an overflow pipe when the screens become clogged, or a sudden rise occurs in the stream. The management of the work on the diverting weir and settling basin was, on the whole, so poor that a statement of the cost would have no especial value.

The Pipe Line.

Development.—The gravity line consists of approximately 7½ miles of 18-in. wood-stave pipe, 3 miles of 16-in. and 1 mile of 14-in. riveted steel pipe. The author was influenced to recommend the use of these materials instead of cast iron by economic motives, involving the consideration of many points, beside the question of first cost; and the wood instead of steel for a variety of reasons, some of which are as follows:

Although there are many examples to the contrary, the author has seen steel riveted pipe of light gauge, designed with a reasonable factor of safety, go entirely to pieces in six years of service. With due regard for proper conditions, on the contrary, he has no doubt that the life of wood-stave pipe is much in excess of that of light gauge steel. Its carrying capacity is very much greater at the beginning, and far more likely to remain practically constant. Its transportation over rough roads is much easier, and the saving in first cost is very great. In this case the saving in first cost over a similar size of No. 12 steel was 43%, and nearly 50% over one of equivalent carrying capacity, 19.15 ins. in diameter. That this advantage, on a basis of equivalent carrying capacity, will be largely increased in the course of years can scarcely be doubted. The cost of any other material would in this case have left no choice with the city but to build a line of much less capacity. The use of the stave pipe will be seen later to have materially influenced the character and details of the location.

Location.—As high a line was located as possible, by following down the divide between the tributaries of the Columbia and Young's River, thus avoiding heavy pressures. The exceeding brokenness of the profile (see Fig. 1) gives some idea of the character of the country traversed, although curvature was very freely used in order to avoid sum-

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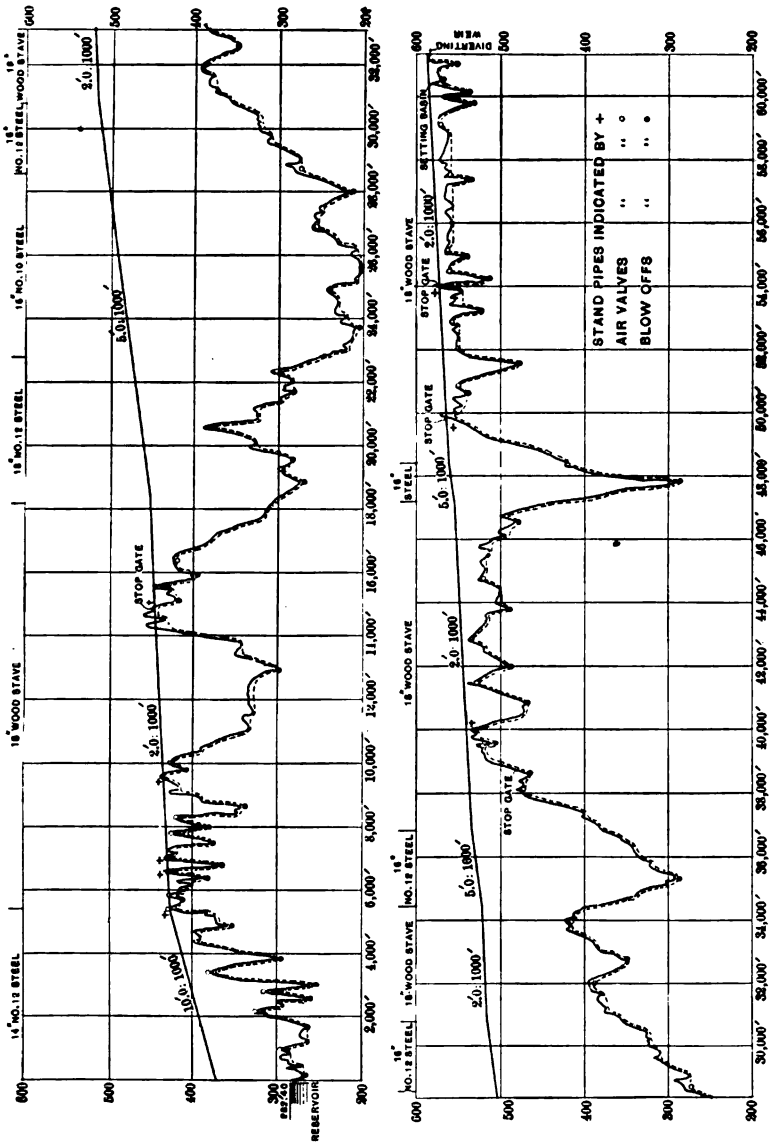

FIG. 1.

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mits and depressions, fully 35% of the line being on curves, either vertical or horizontal. The final line was staked from a paper location, laid on a map of preliminary survey having 5-ft. contours. This method was expensive, but unavoidable in securing a satisfactory line, since the entire distance is through forest, windfall and undergrowth of such density as only those familiar with the Pacific Northwest can form any conception of. Here an instrumentman may consider himself fortunate in finding a place where he can see 25 ft. ahead without clearing, and the stranger, supposing himself to be on fallen logs resting directly on the ground, is often surprised when a misstep sends him plunging down 10 or 15 ft. through ferns, brambles, and among rotten logs to the real ground beneath. Half a mile of line a day with a fully equipped party is excellent progress. The expense of such a method of location was well repaid by a saving of about 1 mile in distance over a very good preliminary. In this connection it may be interesting to note that though the country is so rough, the actual length of the completed pipe is but 0.6% longer than horizontal measurement. One of the most annoying features of the location was the difficulty experienced in securing the land connections with sufficient accuracy for the writing of deeds for rights of way, since much of the country traversed has been laid out into boom-city additions, with lots of 25-ft. frontage, the work having usually been done in the office, without much regard for even correct external boundaries, which were found in the field in almost every case to be entirely at variance with the recorded plats. Many of these additions had been peddled all over the country, entailing the greatest difficulty in securing a satisfactory adjustment of right-of-way matters. By reference to the profile, it will be seen that the location of the wood-stave

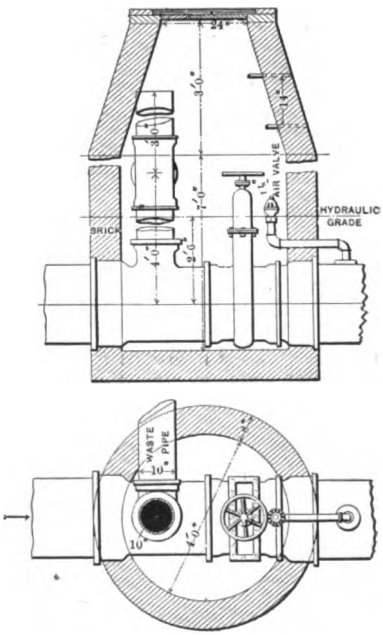

FIG. 2.

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pipe is so made with reference to the hydraulic grade line that at intervals the two approach near enough to permit the use of open stand-pipes (Fig. 2) at these points, while the intervening sections are kept at a lower elevation, causing the pipe to be always full of water, regardless of the quantity flowing, thus realizing the chief essential to long life in stave pipe. At only one point has it been impossible to apply this method, and there the same end is attained by inserting an 18-in. stop gate. By partially closing this when the pipe is carrying less than a full supply, the hydraulic gradient is raised and the preceding summits filled. Danger of over-pressure on the pipe by reckless closing of the gate is entirely removed by the presence on a preceding summit of an open 6-in. stand-pipe. Three hundred feet is the minimum length of radius on horizontal curves, though 60 ft. is frequently employed for vertical curves on the stave pipe.

Pressures.—The maximum pressure adopted for the stave pipe was 150 ft., though this has been exceeded at one point for a few hundred feet, a pressure of 172 ft. being attained. This limit was decided on, not because the stave pipe cannot be made to withstand safely a somewhat greater pressure, but because the author did not wish to depart unnecessarily from conservative practice, and because this was estimated to be about the point of equilibrium in cost between the 18-in. wood and the 16-in. No. 12 steel pipe. The latter was used up to a pressure of 225 ft. head, and No. 10 steel for all in excess, reaching a maximum of 290 ft.

Broken Gradient.—In securing the delivery of the amount of water determined upon, about 4 000 000 galls. in 24 hours, the author adopted the 18-in. diameter for the wood and the 16-in. for the steel, and the incidental broken gradient, in preference to a pipe of uniform diameter, because of the great advantage resulting from being able to use a man on the inside of the stave pipe in the process of building; 18 ins. is about the minimum diameter that can be so handled. The advantage consists in the more perfect rounding out of the pipe to its true form that can thus be secured, and the better inspection of all joints.

Clearing, Road Building and Trenching.—As a consideration for right of way and by special arrangement with the property-owners, the timber and brush were cut for a width of 60 ft., and a 16-ft. road with a maximum grade of 10 ft. in 100 was graded out to the corporate

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limits of the city, a distance of 4¼ miles from the reservoir. Beyond this limit the clearing was made but 20 ft. in width, and only such grubbing done as was necessary for the construction of a narrow road for the delivery of materials and for the trenching. To avoid heavy work in securing suitable grades for the road, all possible advantage was taken of the elevations of the ground for the entire width of the right of way, 60 ft. inside and 33 ft. outside the city limits, the pipe-line location being crossed and recrossed for this purpose. This method entailed much annoyance, difficulty and expense in keeping the road passable after the trench was opened, since much unexpected delay in the securing of materials made it impossible to have them delivered ahead of the trenching. Any other arrangement in such a country was well-nigh impracticable, however, and would have entailed greater expense than that involved in the plan as carried out.

The trenches were staked out for a bottom width of 3.5 ft. for the stave pipe, being widened to 4 ft. on curves, in order to give the room necessary in springing the pipe into line as built. The depth varied from a minimum of 4 ft. to 22 ft. as a maximum, attained in crossing a few sharp points. For the steel pipe, the bottom width was made a foot in excess of the diameter of the pipe, and in all cases the top width was made uniformly 1 ft. greater than the bottom. The material excavated was generally a yellow clay and argillaceous shale, readily moved with mattock and shovel, with occasional short stretches of very soft sandstone requiring the use of some powder to shake it up. The amount of team work in connection with the trenching, other than for back-filling, was very small. Shoring was necessary at but few places, and at these poles cut on the ground answered the purpose.

This work was a part of that undertaken by the contracting firm the failure of which has already been mentioned, and under its management the road work, clearing, grubbing and nearly three-fourths of the trenching were completed.

Cost.—The contract prices were $50 an acre for clearing, $60 for grubbing, 13 cents per cubic yard for road grading, and 17 cents per cubic yard for trenching, including back-filling.

No especial interest attaches to the actual cost of this work by reason of absolutely incompetent management destroying any value that the figures might otherwise possess. It is sufficient to say that

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the cost was greatly in excess of the contract prices. For instance, the actual cost of clearing and grubbing often amounted to fully $800 per acre on a basis of $1 60 per day for labor, while in the trench the men did not average more than 7 or 8 yards a day. The experience of the parties who completed the trenching under the bondsmen of the defaulting contracting firm presents a most forcible example of what efficient management can do, though working at low figures and under most unfavorable conditions. Commencing in October at the beginning of the rainy season, and working much of the time in the mud and rain, this trenching and back-filling was completed at a cost of 17¼ cents per cubic yard with labor at an average price of $1 69 per day, including foreman, nearly 10 yards being moved by each man daily. This cost would have been considerably diminished had there not been much finishing work, very badly scattered, left by the other contractors.

The Wood-Stave Pipe.

Design.—In the design of a wood-stave pipe, the following essential points require consideration: The staves must be thin enough to secure complete saturation and to deflect readily to the degree of curvature employed, and they must be thick enough to prevent undesirable percolation through them. The bands must be of such size that when spaced to secure the desired factor of safety against rupture, there will at the same time be no sensible flexure in the staves and no destructive crushing of the fiber beneath the bands. While fulfilling these conditions, the proportion between the thickness of the staves and the strength and spacing of the bands must be such that the swelling of the wood will not produce injurious strains upon what might otherwise be a properly proportioned band.

The Staves.—The material used for the staves was the native yellow fir. This has been used before for a similar purpose, but the author believes that in each case a much thicker stave was employed. For the Astoria work, the staves were run from 2 x 6-in. stuff with a finished thickness of 1¾ ins., and twelve staves completed the circle. Lumber wholly free from knots, pitch seams and other defects was specified, and the difficulty experienced in meeting the requirements, especially regarding pitch seams, may be understood from the fact that not over 20% of the lumber sawed from selected logs passed final inspection and went into staves. The outcome was a lot of lumber as nearly

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approaching perfection as it is believed has ever been turned out even in the Pacific Northwest. The lumber was about three months from the log when the staves were run, and an allowance of about 2% was made in the width of the staves, to allow for seasoning previous to building and for compression in the process of cinching. Staves were run in lots of about 100 000 ft., B. M., as needed, and formed into the pipe with 6 ins. of dirt over it as soon as possible. They varied in length from 12 to 24 ft., and not over 20% were less than 14 ft. in length. They were run with faces true to the circular form of the pipe, with edges on radial lines and a slight projection or bead along the center line of one edge. For pressures up to 80 ft. head, no special attention was paid to the character of the grain, whether slash grain, edge or quarter sawed; but for all greater pressures, while no increase was made in the thickness, only slash-grain staves were used. This action was the outgrowth of observations made on an experimental section. When put under a pressure of 40 lbs. and allowed to remain some days, with an occasional increase by means of a pump to 120 lbs., a few coarse-grained, quarter-sawed staves allowed a perceptible percolation through them at 40 lbs. pressure, and a considerable leakage at 120 lbs. The selecting of the slash-grain staves entailed practically no additional expense, it being attended to when loading at the wharf for delivery on the line.

Bands.—With staves of this character and bands spaced for a factor of safety of about four, it was determined that a $\frac{7}{8}$-in. round band of mild steel upset to $\frac{1}{2}$ in. would properly meet the requirements regarding crushing under bands and flexure in the staves. Eighty-four thousand were used. Steel was specified of a tensile strength of from 58 000 to 65 000 lbs. to the square inch; a limit of elasticity of 30 000 lbs.; an elongation of 25% in 8 ins., and capable of being bent cold and hammered flat without fracture. The following summary of 58 mill tests made by The Osborn Company may be of interest:

	Elastic limit. Pounds per square inch.	Ultimate strength. Pounds per square inch.	Elongation in 8 ins. Percentage.	Reduction in area. Percentage.
Lowest.....	33 550	59 900	23.75	53.50
Mean.....	40 850	62 166	26.21	63.30
Highest.....	50 660	63 810	30.00	70.90

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The results, as shown by the record of tests and by examination of the broken test pieces, are very satisfactory, but have evidently been secured by the use of high speed in manipulating the machine. Nuts were used about one-fourth thicker than standard. The completed band, before being sent to the work, was carefully coated with asphalt of such a temper and under such conditions that it withstood the great amount of hammering to which these bands were subjected in the process of pipe building without showing any tendency to fly off; and in the accomplishment of this result only one lot of about 6 000 failed to come up to requirements and was recoated. The author has never been able to understand the reason for the difficulty so often reported from the East, in securing an asphalt coating on iron of such temper as to resist shock, and still not rub off in careful handling.

Clips and Shoes.—The tongues used in making the butt joints were of No. 12 B. W. G. steel, $1\frac{1}{2}$ ins. in width and about $\frac{3}{16}$ in. longer than the width of the stave where inserted; they were subjected to the kalameining process to render them non-corrosive.

The saddles or shoes are of the type known as the Allen patent, are made of malleable cast iron of excellent quality, and weigh 10 ounces each.

Strain from Swelling of Staves.—Regarding the last of the requirements for the successful design of wood-stave pipe, that the bands be not overstrained by the swelling of the wood, the author is indebted to D. C. Henny, M. Am. Soc. C. E., manager of the Excelsior Wooden Pipe Company, contractor for this work, for the solution of this matter. Mr. Henny has recently conducted some experiments with very ingenious appliances along this line which are of unusual interest. The experiments consist in making actual measurement of the strains induced in the bands of wooden pipe by the swelling of the staves. For this purpose an ingenious device was designed by Mr. H. Behr, a mechanical engineer of San Francisco. It consists primarily of a very stiff steel spring, resembling a large tuning fork, as shown in Plate I, Fig. 1. A hole is bored through both prongs of the fork near its base, through which the end of the band passes after circling the pipe, the nut being then firmly screwed down. A very slight compressing together of the two prongs, either by tightening the nut or by the swelling of the staves, is magnified at the end of the fork. The motion is further magnified by a simple system of levers and

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transmitted to an index hand revolving on an arc. By means of a testing machine this arc has been graduated to conform to different pressures, so that it constitutes a spring balance designed to measure heavy pressures with quite a high degree of accuracy, and with a very slight deflection at the point of applying the load. With a tension of 10 000 lbs. on the band to which the spring is attached, the deflection is but 0.02 in. Two experiments made on independent sections of stave pipe built of kiln-dried yellow fir staves 1⅜ ins. thick, each section being 18 ins. in diameter and 12 ins. long, banded with ½-in. round bands, were conducted in the following manner and with the following results :

Section 1.

No. 1. On February 27th, a strain of 4 750 lbs. was applied and the pipe immersed. The following strains were recorded on succeeding dates :

Date.....	March 1st.	March 4th.	March 7th.	March 9th.	March 12th.
Strain, lbs.	3 400	2 950	2 750	2 525	2 525

The final strain maintained between the staves was 153 lbs. per square inch of contact.

No. 2. The band was loosened to the strain indicated and immersed again with the following results :

Date.....	March 12th.	March 18th.	March 21st.	March 26th.
Strain, lbs.....	100	1 500	1 500	1 550

The final strain maintained between the staves was 94 lbs. per square inch of contact.

No. 3. The band was loosened again to the strain indicated :

Date.....	Mar. 26th.	Apr. 1st.	Apr. 4th.	Apr. 8th.	Apr. 11th.	Apr. 16th.
Strain, lbs.	500	1 000	1 050	1 100	1 200	1 225

The final strain maintained between the staves was 74 lbs. per square inch of contact. The staves weighed 18 lbs. dry and 27¾ lbs. after the experiment.

Section 2.

No. 1. The band was first cinched up to a tension of 5 000 lbs., then loosened to the strain indicated, and the pipe finally immersed:

Date.....	Apr. 16th.	Apr. 17th.	Apr. 20th.	Apr. 24th,	Apr. 27th.
Strain, lbs...	450	1 400	1 900	2 000	1 975
Date.....	May 1st.	May 4th.	May 7th.	May 10th.	May 13th.	May 17th.
Strain, lbs.	1 900	1 900	1 800	1 700	1 600	1 600

The final strain maintained between the staves was 97 lbs. per square

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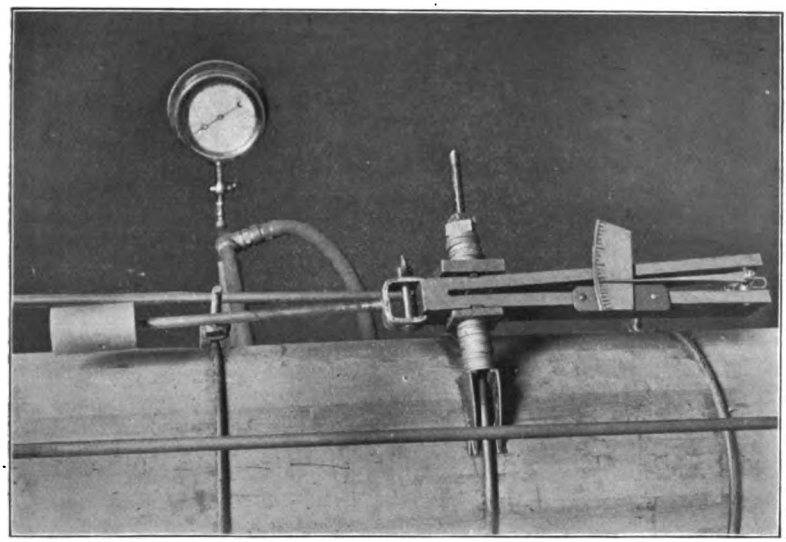

FIG. 1.

FIG. 2.

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inch of contact. The staves weighed 26 lbs. when the experiment was finished, and 20.8 lbs. after being kiln dried subsequently.

These results present an interesting study, but, without enlarging on all they set forth, it is plainly shown that the maximum compressive strain the wood can resist permanently is not much in excess of 150 lbs. per square inch. This fact, since the pressure between the staves must considerably exceed the internal pressure tending to force them apart in order to maintain a tight joint, demonstrates conclusively the limit of safe pressure in wooden pipe construction to be little if any in excess of 100 lbs. per square inch under the most favorable conditions with this class of timber. They also show that when the initial strain is very small the swelling of the staves alone is capable of developing a temporary strain of about 125 lbs. per square inch, a fact which will readily account for the frequent bursting of tanks built with thick staves, and small factors of safety. It will also be observed that, while the pressure is largely self-adjusting, whether a large initial strain is applied in back cinching before filling or the bands but lightly cinched, the former practice seems to insure a somewhat greater permanent pressure between the staves and a correspondingly tighter pipe. So far as these results influence the spacing of bands in pipe construction, they need only be regarded when dealing with low heads and wide spacing, the strains then arising from this source being larger relatively to the bursting pressure due to water head than under any other conditions. In close spacing, the strain from this source falls, of course, below that ordinarily allowed for cinching.

The band spacing employed was as follows:

Head in feet.	No. of bands per 100 ft.	Spacing in inches.	Head in feet.	No. of bands per 100 ft.	Spacing in inches.
0 to 25	100	12	87 to 92	282	4¼
25 " 30	110	11	92 " 98	300	4
30 " 35	120	10	98 " 101	309	3⅞
35 " 40	134	9	101 " 105	320	3⅝
40 " 45	142	8½	105 " 109	331	3⅝
45 " 49	150	8	109 " 113	343	3½
49 " 52	160	7½	113 " 117	356	3⅜
52 " 56	172	7	117 " 121	370	3¼
56 " 60	185	6½	121 " 126	384	3⅛
60 " 65	200	6	126 " 131	400	3
65 " 68	208	5¾	131 " 137	418	2⅞
68 " 71	219	5½	137 " 143	437	2¾
71 " 75	228	5¼	143 " 150	457	2⅝
75 " 79	240	5	150 " 158	480	2½
79 " 83	253	4¾	158 " 166	506	2⅜
83 " 87	267	4½	166 " 175	534	2¼

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Construction.—J. D. Schuyler, M. Am. Soc. C. E., has so fully described the general method of erecting this class of pipe in his paper on the Denver Water-Works* that it is unnecessary to elaborate it here. A few points only are worthy of mention. Yellow fir, being a hard and rather unyielding wood, tight cinching was the invariable practice; no butt joints were allowed to pass unless fully driven home; bands, shoes and nuts were repainted with asphalt or paraffin paint immediately before covering; back-cinching followed erection on the following day, and back-filling to a depth of 6 ins. over pipe as soon after as possible; all openings in the pipe over 4 ins. in diameter, beside the 4-in. open stand-pipes, were made with special castings and oakum joints 7 ins. in depth, while smaller openings were made by bolting on saddles over lead gaskets, and placing two bands over the top.

Since it is a matter of the utmost importance to the success of this class of work that it be not undertaken by an inexperienced contractor, the specifications excluded the bids of all parties not able to submit satisfactory evidence of their skill and experience in the construction of stave pipe under heavy pressures. Proposals were invited, and payment made on the basis of pounds of steel used in the bands and feet board measure used in the staves, the price paid for these items representing compensation in full for all expenses involved in supplying these materials and the erection of the pipe complete, exclusive of fixtures, and the back-filling to a depth of 6 ins. over the pipe. The contract was awarded to the Excelsior Wooden Pipe Company, of San Francisco; and to the large experience and hearty co-operation of the company's efficient manager, D. C. Henny, M. Am. Soc. C. E., is due in no small degree the unqualified success of the pipe-laying. This commenced in the latter part of July under some disadvantages, but by aggressive and capable management the work was hurried through to completion in October before the heavy rains had set in.

Tightness.—A test for tightness was made of the upper 2½ miles of this line shortly after the water was first turned in. This gave results which the author believes have never been surpassed in any other pipe construction of any class. The pipe was filled from the head works, and, the gate at the lower end of the section being closed, the water rose and passed off through the overflow from the stand-pipe immediately

* See *Transactions*, Vol. xxxi, p. 135.

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adjoining the gate. The head gate was afterward closed. This gate was not absolutely tight, but permitted the passage of a little trickling stream, not exceeding, perhaps, 1 quart in a minute. The assistant engineer in charge of the work was much surprised on the following day to observe this same little trickling stream, apparently undiminished in quantity, passing through the waste pipe at the end of the line. The pipe, contrary to the usual experience with stave pipe, was entirely tight from the beginning, which condition the author largely attributes to hard back-cinching, and the probable absorption of moisture from the damp back-filling. This particular piece of pipe was the only one on the line which permitted a determination of its tightness to be made independently of the steel pipe.

Cost.—The contract prices of the two chief items of this work were: Steel in bands, 4.8 cents per pound; lumber, feet B. M. in staves, measured before milling, $35 40 per M. The average spacing of the bands is $5\frac{2}{16}$ ins. The cost to the city, including all appurtenances, was 90.33 cents per foot, and 76 cents excluding such appurtenances. The whole amount of the contract was $36 100, and the total extra work cost $29 35.

The actual cost of the work the author does not feel at liberty to give by reason of the large personal interest of certain contractors being closely allied thereto. The distribution of the cost was as follows: Building and placing bands, 55%; back-cinching, 26%; repainting iron work, 3%; back-filling to depth of 6 ins. over pipe, 8.75%; placing specials, 3.5%; placing air valve, 0.75%; unclassified labor, 3 per cent.

The rate of wages paid for 10 hours was $1 75 for common labor and an average of $2 71 for foremen. It is presumable that the contract prices represent a profit of from $12\frac{1}{2}$ to 15 per cent.

The Steel Pipe.

Quality of Steel.—The specifications for the sheets used in the manufacture of the riveted steel pipe were the same as those for the bands of the stave pipe, with the exception that the test pieces were to be $\frac{3}{8}$ x 2 ins., and it was to be made by the open-hearth process.

Manufacture.—The sheets were 4 ft. in length. Alternate large and small courses were used, the small courses having the full nominal diameter of the pipe. The last of the eight courses constituting a length of pipe was made slightly conical, so that, being a small course

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at the beginning, it was expanded to the size of a large course at the end. The addition of this course was a concession made to the contractor in order to constitute a saving in lead, the specifications requiring a large course at each end, and the addition of two courses making the sections too long for convenient handling or laying. In reality, little advantage was gained by the addition of this eighth course, owing to the difficulty of handling pipes of such length and making connection between them in the trench when using this conical course, and because of the more frequent necessity for cutting pipe in rounding curves. All straight seams were double riveted, and round seams single riveted, the seams being proportioned after Professor Kenneday's formulas for the use of iron rivets. The plates were punched from the sides coming together in the lap. The No. 12 steel was chipped and calked about the union of straight and round seams only, while the No. 10 sheets were beveled, and all seams calked with pneumatic calkers. Each length of pipe was required to pass through the testing machine, and to be tight under the following pressures: 14-in. pipe of No. 12 steel, 200 lbs. per square inch; 16-in. pipe of No. 12 steel, 175 lbs. per square inch; 16-in. pipe of No. 10 steel, 230 lbs. per square inch.

The author had some doubt as to the possibility of making riveted pipe of such light steel perfectly tight before dipping, under the pressures specified, and the manufacturers objected most strenuously to testing every piece.

In practice it was found that by putting every piece through the test, all spurting streams could be eliminated. Mere dropping was not regarded as serious in view of the coating still to be applied, and could not be wholly prevented. By insisting upon strict compliance in the matter of test, it is believed that very superior work was secured; such results as to tightness could scarcely have been secured, however, had not red lead been used in the lap on all joints. After test, the pipes were subjected to an asphalt bath applied under the customary conditions, and a most satisfactory coating was secured. By actual experiment it was found, after a very leaky pipe was coated, it could not be made to leak under any pressure short of bursting.

Joints.—The joints were made by the use of welded iron sleeves $\frac{3}{8}$ in. in thickness, 6 ins. in width, and having a lead space of about $\frac{3}{8}$ in. A reinforcing thimble of No. 8 steel, 8 ins. in width, was inserted half its width at the shop and riveted in one end of each pipe.

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Oakum was only used to fill the crack at the junction of abutting pipe, none being used in the joint proper, the annular space being run entirely full of lead.

Cost.—Nothing of special interest attaches to the laying of this pipe, and by reason of inefficient management, coupled with much rainy weather, the cost to the contractor just about equaled the price received, which included laying and back-filling 6 ins. over the pipe. The price per foot paid by the city and the cost of the pipe to the contractor at the wharf were as follows:

Size.	Contract price.	Cost of pipe to contractor.
14-in.	No. 12.. $1 10	$0.85
16-in.	No. 12.. 1 18	.90
16-in.	No. 10.. 1 38	1.00

Pipe Line Fixtures.

Blow-Offs.—On the 18-in. pipe the blow-offs are all 6 ins., and on the smaller sizes, 4 ins. in size. They are provided with flanged gates and tangential connections, and are riveted to the steel pipes, while special castings are used for the stave pipe.

Overflow Waste.—A simple combination of stop gate with a relief overflow and stand-pipe is located on certain summits, by means of which the water can be partially or wholly turned off without in any way interrupting the continuity of the flow in that part of the line above the gate, thus rendering any overstraining of the pipe from undue pressure an impossibility. The height of the hydraulic gradient is also rendered adjustable within certain limits, whereby the stave pipe may be kept filled at all stages of flow. The stand-pipe also serves to admit or release air from the summit on which it is located, as may be required. An air valve connects with the pipe below the stop gates, for the purpose of admitting air when the water is drawn off while the gate is closed, as shown in Fig. 2.

Air Valves.—The air valves are the invention of Mr. Thomas W. Brooks, of San Francisco, and have been but recently brought into use. The author desires to testify to their satisfactory performance. They are made with two types of interchangeable valves; one is a wooden ball with vulcanized rubber covering, and the other is a metallic valve having a concave lower surface. They remain open by their own weight until closed by the internal pressure; and are in-

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stantly opened by any tendency to a vacuum within the pipe. They require practically no attention, and the water may be turned in or out of the pipe line with impunity, and without fear of interruption from air, either from within or without. An angle valve operated by hand provides for the release of any air accumulating while the line is in operation. The air valves have been arranged in groups of several of small size instead of one of large size, and are so proportioned that a sudden breaking of the pipe at the most dangerous points will not bring an atmospheric pressure on the pipe in excess of about 8 lbs. A suitable stop gate makes possible the removal of any of the valves without interruption to the flow.

Open stand-pipes are used in preference to air valves where practicable.

Suitable manholes of brick masonry, with wooden covers well fastened on, are built over all air valves, blow-offs and stop gates.

Pipe Line Experiments.

Capacity of the Pipe Line.—Just previous to the determination of the leakage in the reservoir, an 18-hour test was made to determine the delivering capacity of the pipe line. The measurement was made in the reservoir, it being empty at the beginning. Observations were taken of the height of the water surface once during the test, and again at its close. The results in the two cases showed a constant rate of inflow. The results computed from the total inflow were as follows:

Volume inflow in 18 hours.....	3 077 170 galls.
Rate " " 24 "	4 102 893 "
Add for loss by reservoir percolation.....	13 560 "
Total capacity of line in 24 hours.....	4 116 453 "

The loss by percolation is assumed to be constant for all depths, which assumption is, of course, not strictly correct, but the error is so small that its practical importance is nil. The rate of delivery promised the Commission was 4 000 000 galls., the amount made the basis of calculation was 4 050 000. The excess of delivery over the amount assumed in computation is about 1.6 per cent.

Determination of Leakage in Pipe Line.—To determine the amount of loss in the pipe line, the water was turned out of the settling basin until the indicated flow over the weir and into the pipe line was at the uniform rate in 24 hours of 326 690 galls., less a percentage of error in

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the weir of 9 147 galls., making the net inflow at rate of 317 543 galls. The rate of flow at the lower end of the pipe line, after a lapse of sufficient time for the water to attain its uniform minimum flow, was determined by measuring it in the steel tank in the gate well. The following results were secured:

	To top of second course.	To end of experiment.
Depth of water.....	9.19 ft.	17.11 ft.
Volume.....	447.92 cu. ft.	843.45 cu. ft.
Time of filling, in minutes.....	19.18	36.10
Rate of inflow in 24 hours.....	251 540 galls.	251 660 galls.
Add for average leakage through gates of tank.....	1 410 "	1 410 "
Net rate of pipe line discharge..	252 950 "	253 070 "

Net rate of pipe line discharge taken at 253 010 galls.

Apparent loss in line 317 543 — 253 010 = 64 533 galls.

In view of the fact that observations on a long section of the stave pipe when first filled showed it to be entirely tight, and that it has been necessary to correct quite a number of leaky lead joints on the steel pipe, even in places where the pipe was put under pressure and made entirely tight before covering, as was the case with almost all of it, it is believed that this loss must be accounted for by the existence of leaks of some magnitude in these lead joints, which the saturated condition of the ground at this season of the year has prevented being yet discovered. However the percentage of loss is believed at present to be smaller than is usually obtained in lines of similar length.

Determination of Frictional Loss in Pipe Line.—The existence of the open stand-pipes on the line afforded an excellent opportunity for the accurate determination of the loss of head between them. The following observations were made when the line was discharging its full capacity, 6.369 cu. ft. per second.

Station.	Actual Length.		Observed height in stand-pipe.	Elevation, computed.	Fall per 1 000 ft.
	Stave.	Steel.			
540	.....	.....	572.61	572.65	
	4187.66	.....	.....	.....	1.9628
498.35	.....	.....	564.39	564.30	
	19130.56	.....	.....	.....	1.9628
144.25	.....	16416.38	.....	.....	5.0023
	.....	.....	444.72	444.50	

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These are all the measurements which have been completed up to the present time, with the exception of an attempted determination of the fall in the hydraulic grade line between the settling basin and the stand-pipe at station 540. The result was a verification of that obtained between stations 540 and 498.35, but is omitted by reason of an uncertainty amounting to a few tenths of a foot in the entry head at the basin. It will be noticed that the fall used on the hydraulic grade line in construction was 2 ft. per 1 000 for the stave pipe, and 5 ft. per 1 000 on the riveted steel pipe. The almost exact coincidence between these figures and those actually assumed by the flow in the pipe is certainly remarkable, especially in a pipe line so frequently compounded. and where curvature is so freely used. The differences between actual and computed heights of water in the stand-pipes will be seen to vary from 0.04 to 0.22 ft., differences which are quite within the range of probable error in average level work. It will also be noticed that the greatest variation, 0.22 ft., occurs at station 144.25 after the water has passed from the previous stand-pipe through four different sections of 18-in. stave pipe and three sections of the 16-in. steel, in all aggregating about 7 miles. The actual frictional head per 1 000 ft. in the stave pipe is seen to be 1.9628 ft. In determining the loss of head in the steel sections, this value is assumed for the stave pipe between stations 498.35 and 144.25, making the frictional loss in the steel 5.0023 per 1 000 ft.

Substituting these results in the Chezy formula for the stave pipe, $c = \frac{v}{\sqrt{r\,s}} = \frac{3.605}{0.02713} = 132.88$, from which the value of the frictional coefficient n of the Kutter formula for c is deduced to be about 0.00985; and for the steel pipe:

$$c = \frac{4.584}{0.04083} = 112.3 \text{ from which } n \text{ is about } 0.0111.$$

Engineers will not fail to note the large advantage in small frictional loss possessed by the stave pipe over the steel, and to observe that the value of 0.010 for n, used by many engineers in dealing with stave pipe, is here found to be practically correct.

The author commends the result obtained from the steel to the consideration of those engineers who advocate tremendous allowances for friction in pipe of this character compared with those for cast-iron pipe; and as a whole, the results obtained, to the consideration of other engineers who have held that large shortages in delivering

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capacity of gravity pipe lines may be accounted for by failure to avoid up and down hill travel, and in not accepting the alternative of heavy pressures and proportionally increased cost incident to more uniform grades on valley routes. That the present discharging capacity of the steel pipe will remain constant through a period of many years the author does not expect, while the almost perfect condition of the inside surface of the wrought-iron pipe taken up from the original pipe line, after carrying Bear Creek water for twelve years indicates that there will probably be no very serious falling off.

The Reservoir.

The reservoir has a capacity to the level of the overflow pipe of 6 250 000 galls., and has a depth of 17.4 ft. to the flow line, and 20 ft. to the top of the parapet wall. The general shape is that of the hill on which it was built. The embankment is quite uniformly about 5 ft. in height on the inner slope, except at one point where it is 15 ft. in height. The remainder is in excavation. The top of the bank is about 23 ft. in width; and varies from 0 to 35 ft. in height above the toe of the outer slope. Both inner and outer slopes were intended to be 2 to 1, but the disposal of a considerable quantity of surplus material left the outer slope in places somewhat steeper than this.

Excavation.—The materials in which the excavation was made consisted of much black indurated clay, and a fine grained sand mixed with clay, which in places approached soft sandstone in character. All material was plowed, and when broken was in excellent condition for handling with wheel scrapers. When excavated, it possessed just the right degree of moisture to solidify to the best advantage without the addition of water. All surface soil containing roots or other vegetable matter was first removed, and care taken to secure a proper bond between the natural ground and the made earth. The banks were carried up in layers 6 ins. in thickness, and were thoroughly rolled with a smooth roller weighing 3 000 lbs. to the lineal foot. On the inner slope the bank was made 1 ft. wider than the finished work, and afterward dressed to the proper plane. By the use of slope boards and lines, the whole inner surface was dressed very truly to the desired form without any after-filling being necessary. The price paid for the excavation, 15 cents per cubic yard, included the clearing of the site, previously covered with burned timber, and the construction of the embankments.

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The cost to the contractor, 16.9 cents, was needlessly large, as the haul was made unnecessarily long during much of the work, and a scarcity of ready cash made it difficult for the contractor to get rid of worthless men. The total amount of excavation was 49 540 cu. yds.

The Lining.

On the bottom, the reservoir is lined with 6 ins. of concrete laid with expansion joints, $\frac{3}{8}$ in. of cement mortar, one coat of liquid asphalt, and one asphalt coat of harder consistency. The slope lining consists of 6 ins. of concrete laid with expansion joints, one coat of asphalt, one layer of brick laid flat after dipping in hot asphalt, and a final finishing coat of asphalt.

The parapet wall surrounding the reservoir is 2 x 4 ft. in section and built of basalt, with rough rubble backing and squared face stones. It is surmounted with a 9-in. course of coping stone, on which is erected an iron picket fence 5 ft. in height.

General Design.—In the design of this reservoir the author endeavored to secure a lining which would at a reasonable cost fulfil the following somewhat ideal conditions: It should not readily break down over slight local settlements; if it did break down the impervious surface should bend down and not break. It should not crack from expansion and contraction, but if it did crack, no leakage should occur. It should be permanent, and, finally, it should be practically water-tight. The lining adopted is believed to meet all of these conditions. The author has tried reservoir linings made by laying brick dry on a layer of coarse underdrained gravel, and then coating it with asphalt; of asphalt mastic covered with asphalt, of brick laid in cement mortar and coated with asphalt, and of asphalt directly on concrete. All have proved sufficient for the purpose, and some of them especially useful as expedients where large results were required from small expenditures, but he feels that the lining adopted for the present work more nearly meets all the requirements of an ideal reservoir lining than any other with which he is familiar. The cost per square foot, without profit, was as follows:

Slope.	
6 ins. concrete	$0.1187
First coat asphalt 0.649	0.01005
Brick in asphalt	0 08886
Second coat asphalt 0.851	0.01307
Chinking crevices	0.00300
Ironing	0.00356
Total	$0.23724

Bottom.	
6 ins. concrete	$0.10311
Cement mortar finish	.01126
First coat asphalt 0.537	.00768
Second coat asphalt 0.573	.00821
Total	$0.13026

Average cost for the entire reservoir, $0.17965 per square foot.

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Concrete.—The concrete is composed of crushed basalt rock, 2½ ins. in maximum size, quarried and crushed near the work; Columbia River gravel, a very clean fine black gravel containing much coarse sand; fine sharp sand, and imported Portland cement. One cubic yard of concrete contained 0.9 cu. yd. of crushed rock, 0.5 cu. yd. of gravel, 0.1 cu. yd. of sand, and one barrel of cement.

All mixing was done by hand on movable platforms in ½-cu. yd. batches. Three gangs were usually employed, each having six mixers, who also placed the concrete for the two men engaged in finishing and tamping. The coarser rock were raked forward and down, leaving the finer material at the surface, and a long straight edge was used in securing a uniform surface. All three gangs were served by nine wheelbarrows, five handling rock; three sand and gravel, and one, cement. A helper at the cement, one man tending water, one sprinkling concrete already laid, and the water boy, made up the concrete force. The sand, gravel and cement being first thoroughly mixed, water was then added, mixed and spread, and the wet stone added. It was then turned three times on the mixing platforms, and once more when deposited. On the slopes a rough finishing coat was applied as soon as the batch was in place, by the application with a shovel of a little mortar taken from the next mix. A very good surface, smooth and free from stones was thus secured for the laying of the brick, while the comparative roughness of the surface gave the asphalt coat a very perfect adhesion.

On the bottom, the ¾-in. finishing coat of cement mortar was applied as fast as the concrete was placed by two finishers using smoothing trowels, who were served by four men mixing and carrying. The mortar was mixed in the proportions of two to one of fine sand and cement. In all concrete work no more water was used than was necessary to mix properly, only a very little showing on the surface when tamped. When the ground was hot and dry, water was sprinkled on it in advance of the concreting. On the slopes the concrete was placed in sheets 10 ft. wide, extending up and down the slopes. On the bottom it was placed in squares measuring 20 ft. on a side. These boundaries were maintained when placing concrete by means of a 2 x 6-in. plank, the straight edge being used between these in securing a uniform surface. When connection was made between sheets the 2 x 6-in. plank was replaced with a piece of beveled ½-in. ordinary weather boarding,

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4 ins. in width. After the concrete had thoroughly set, about two weeks after placing, these strips were removed, and in making application of the first coat of asphalt, the grooves were run full. There was no difficulty in removing the beveled strips, but some strips with vertical sides and an occasionally inverted beveled one proved exceedingly troublesome. Plate I, Fig. 2, illustrates the appearance of the reservoir during the progress of the concrete work.

Expansion Joints.—These expansion joints have fulfilled every expectation. Not a crack has appeared anywhere in the concrete. Their operation was very noticeable during warm weather, especially in the bottom. In the cool early morning they were much depressed, while in the heat of mid-day they presented the appearance of ridges discernible clear across the reservoir, the amplitude of the motion being in many cases fully $\frac{3}{4}$ in. Observations made by the author on a large concrete-lined reservoir, recently built without any provision for expansion and contraction, in which cracks had formed up and down the slope for a distance of several hundred feet at remarkably uniform intervals of about 25 ft. leads him to think about 20 ft. a suitable maximum spacing for expansion joints in concrete laid in this climate. By reason of its superior elasticity the author considers the use of pure asphalt of medium hardness much superior to any mixture of asphalt and sand for this purpose, with suitable provision for holding it on the slope, and doubts the efficiency of a mastic unless made very rich in asphalt.

Cost of Concrete.—There were 603.3 cu. yds. of concrete used on the slopes, which cost the contractor $1.072 a yard for labor and $1.939 for materials, except cement, which was furnished by the city, and cost $2.822 per yard of concrete. The amount of concrete on the bottom of the reservoir was 678.2 cu. yds., and cost the contractor $0.674 per cubic yard for labor and $1.923 for material, except cement; this was furnished by the city and cost $2.641 per yard of concrete. The contract price was $3 50 per yard, exclusive of cement. The total cost of the cement coating was $1.126 per 100 sq. ft. That the work was well managed is apparent from the fact that 1.84 cu. yds. of concrete was mixed and placed each day for each man employed. On the bottom alone, this amount was increased to 2.35 cu. yds. For work of this character the author considers this a creditable showing, for he has known instances where the labor account amounted to four times as much on

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work of a similar character but larger proportions. For most of this work Italians were employed who were experienced in concrete work. The crushed rock was secured from a quarry located about 800 ft. from the reservoir, at about the same level, and delivered on the work at a cost of 95.28 cents per cubic yard. The sand and gravel were dredged from the Columbia River and delivered by sub-contract at the wharf at a cost of 86.59 cents per cubic yard.

Cement.—The city purchased by contract all the cement used on every part of the work at a cost of $2 45 per barrel delivered at the wharf, as needed on the work. In all about 3 100 barrels were used, of which 1 558 went into the reservoir lining. Most of this was of a superior quality. Every sixth barrel was tested for tensile strength, with frequent tests for blowing, and time of setting. Only seven barrels were rejected, these having deteriorated by exposure to weather.

The Asphalt Work.—The asphalt used in the work was all of the Alcatraz brand. Two grades were employed; the L (liquid), and XXX (paving cement). The first is a natural liquid asphalt, and the second is the product of refining the natural rock asphalt with about 20% of the liquid as a flux. Both grades are now placed upon the market in barrels holding about 400 lbs. As a rule no asphalt was applied to the concrete until the latter had been in place fully two weeks, and was well dried on the surface, though some slight variation from this rule was unavoidable by reason of the work being overtaken by the fall rains before completion. On the bottom was first applied a coat of the L grade, which was followed by a coat of the XXX paving cement. On the slopes none of the L grade was used by reason of the tendency to creep, which a soft underlying coat always produces, and so often mars the appearance of work on reservoir slopes. The author, after a considerable experience in using asphalt in different ways as a reservoir lining, is disposed to believe that any advantages possessed by a soft coat over a harder one, as a first application, is more fancied than real. At the proper temperature, the harder grade runs just as readily, and enters all crevices just as surely when applied, and if the masonry be entirely dry and clean, and preferably a little rough, it adheres more tenaciously than the liquid. The only superiority possessed by the latter as a first coat is that if it must be applied on a damp surface, it will adhere where the other will not.

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All dust was carefully swept from the concrete, as it effectually prevents any bond, and the asphalt was applied with mops made from twine used in the local salmon fisheries. The mops were served from large sheet-iron buckets, which were kept filled by attendants, who carried asphalt from two large kettles holding about 3 000 lbs. each, in which it was melted and kept at the proper temperature. The bricks used on the slopes were about half vitrified and half common, the contractors being disappointed at the last moment in the delivery of the requisite number of the former. As laid they were submerged in a bucket of hot asphalt, and placed in position on the slope by means of iron tongs. Sufficient asphalt always streamed from the brick to fill entirely all irregularities beneath it. A push joint was made, much as when laying brick in cement mortar, but much more expeditiously. For the sake of close joints and consequent economy in the use of the asphalt, it proved very important to keep the temperature high enough for the asphalt to run like water, which is the manner in which it should always be used to secure the best results. Common laborers could, with a little practice, when well supplied with materials, readily lay an average of 2 300 bricks in 10 hours. The finished coat of asphalt followed as speedily after the brick laying as possible, in order to avoid delay from rain, as the water standing in unfilled joints dried out very slowly. In applying this coat an effort was made to fill all joints, and where this could not be done with hot asphalt on a slope, a rich mastic of sand and asphalt was used. This much improved the appearance of the work, as the brick were quite rough and somewhat irregular in shape. The connecting joint between the top of the lining and the base of the parapet wall was also run full of asphalt. A final smoothing was given the finished surface by going over the slopes with hot irons. Although the general appearance of the surface was by this means much improved, the author is rather loath to trust hot iron in the hands of common laborers on an asphalt surface. Unless most carefully watched, overheating and consequent serious injury are almost certain to occur. When this lining had been exposed to the rays of the sun for a considerable time, there was a very noticeable sliding of the brick on the slope, and a consequent closing up of the joints, crowding out the asphalt where they were thick by reason of the asphalt having been used too cold on days that were too windy to permit it being

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maintained at a proper temperature. As a consequence, openings were produced between the brick lining and the wall at the top of the slope, which were filled with mastic. The footing course of brick was prevented from sliding by being set into the concrete of the bottom lining. After a few weeks all motion ceased entirely.

In order to secure the best results possible, all asphalt work should be performed during the dry summer months. Such was the plan for the Astoria work, but the delay in beginning carried the date of completion over into the rainy season. By careful management in keeping the work in proper shape for speedy drying, no especial difficulty was experienced except with the brick. Although they were housed at the reservoir, and piled and fired when rained on during transportation, sufficient moisture was apparently absorbed from the air to produce by evaporation small bubbles or blisters on the surface when dipped in the hot asphalt. No similar difficulty was experienced with brick laid during the dry weather. This defect is one of appearance rather than reality, and is but temporary, as the steam soon condenses and the bubble collapses. Although the adhesion between the brick and the asphalt is sometimes destroyed over the limited area covered by these bubbles, the effect is simply to permit these brick to become more quickly saturated than would otherwise be the case. It must not be assumed, as has been so often stated, that asphalt renders anything wholly impervious to water, or is of itself impervious. On the contrary it may be readily demonstrated by a simple experiment that a brick thoroughly coated and recoated will in time absorb as much water as an uncoated one. The advantage of the asphalt lies in its retarding effect on the passage of water ; it does not exclude it.

Cost of Asphalt Work.—Tables Nos. 1 to 7, inclusive, showing the amount, cost and distribution of both labor and materials employed in the asphalt work, are prepared in a way that is believed to make the probable cost of other similar work easily determinable. In view of the interest which the use of this material as a reservoir lining is very properly creating among engineers, especially in the East, this presentation may not be untimely.

Iron Fence.—On top of the coping which surmounts the parapet wall is erected an iron picket fence 5 ft. in height, having pickets ½ in. square, spaced 4 ins. between centers. The rails are small channel irons 2 ins. in width. No attempt is made at ornamentation. The

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panels are 8 ft. in length, and at the end of each a post picket extends 4 ins. into the stone. Each post is braced from the face of the wall, the brace also having a tie fastening into the top of the wall, making three drilled holes necessary for each panel. The panels are supported half way between the posts. Both supports and braces are made adjustable by means of movable clips and set screws, rendering possible the securing of perfect line and grade. All joints were made with lead and calked. This fence was designed and made by the Van Dorn Iron Works, of Cleveland, and was erected by day labor under the supervision of one of the assistant engineers, without a foreman, at a cost of $985 40 for a total length of 917.2 lin. ft.

Aerating Fountain.—At the center of the reservoir is a 12-in. entry pipe leading from a connection with the gravity conduit in the power house, and passing up through a concrete pedestal. It terminates a little above the surface of the water in a brass fountain head, having about 300 holes, ranging in size from $\frac{1}{16}$ to $\frac{9}{16}$ in. in diameter, arranged in concentric rings with the larger at the center, and gradually diverging from the perpendicular to an extreme angle of 35° on the outer circle. Through this the water may be turned from the pipe line, when not used for the development of power, and the reserve head may be used in the formation of an aerating fountain. Its construction was a natural consequence of having a reserve head for power development. It proved a very ornamental feature, and is expected to be useful in securing aeration and free circulation in the water, which is necessarily heavily charged with vegetable matter, and, when allowed to stand in a reservoir without circulation for a considerable time in the summer, has developed very offensive qualities.

At the foot of the fountain pedestal, in the 12-in. supply pipe, is a stop gate, and a double nozzle fire-hose connection for the purpose of securing streams for cleaning out the reservoir.

Total Cost of Reservoir.—The total cost of the reservoir, exclusive of iron fixtures, which are included in the cost of the gate and power house, is summarized in Table No. 8.

Reservoir Leakage.—When first filled, careful observations were made to determine the rate of leakage from the reservoir. The depth of water was about 14 ft.; the time elapsing between the observations was 68 hours. As the air was in a state of saturation all the time there was no occasion to make deductions for evaporation. The rainfall

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record was taken from the reports of the local signal service. The results were as follows:

Total rainfall..... 7 384 cu. ft.
Total quantity retained ..... 2 253 "
Total leakage..... 5 136 "
Total leakage in gallons in 24 hours..13 553 galls.
Total leakage in gallons in 24 hours per 1 000 sq. ft. of lining submerged, 232.1 galls.

Assuming the absence of evaporation, and that the rate of leakage is uniform for all depths, it would require a term of years for the reservoir to empty itself in this way, since the average rainfall is 75 ins. For a new reservoir filled for the first time, and with consequently no opportunity for sedimentation, the results are gratifying. A second experiment of similar nature, made a few days later, gave results identical with the preceding.

The Power and Gate House.

There is a surplus fall in the mile of 14-in. steel pipe at the end of the conduit, which is to be used for the development of power by Pelton wheels for the operation of an electric plant for the illumination of the streets and public buildings, and for pumping a water supply to the districts not capable of being supplied directly by gravity.

The Building.—The gate house for controlling the water at the reservoir and the power house for the light and pumping plant have been united in one building of basalt masonry, with cut face stones laid random, and rubble backing. It has a copper roof, is sealed throughout the lower story with red wood, finished in the natural grain, and has three living rooms finished off upstairs for the residence of the keeper. The end enclosing the gate well is semi-circular in form. The building as a whole presents a pleasing effect, which should be credited to Mr. J. E. Ferguson, the architect.

Arrangement of Power-House Fittings.—The arrangement of the power-house appliances may be readily understood from the plan (see Fig 3). The water passes directly through the building to the fountain in the reservoir. When the power is utilized for work, the fountain is cut out and the water discharged through the nozzles of either of the Pelton wheels. The tail water is conducted the length of the building

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in a concrete channel under the floor and discharged into a 14-in. pipe leading into the gate well. It will also be noticed that provision is made for connecting the suction of the pump to be installed later directly to the pipe under the same pressure as the nozzle operating

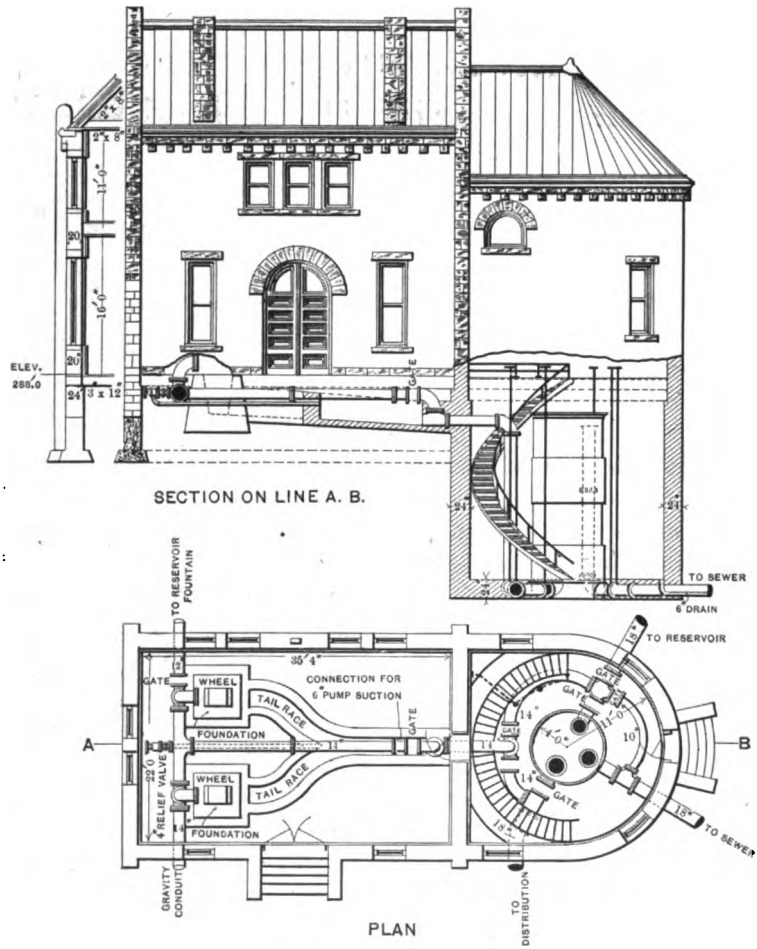

FIG. 3.

the wheel which is to drive the pump, thus diminishing by that amount the work necessary to be done in elevating the desired amount of water, which will never exceed 200 000 galls. in 24 hours. When the flow at the power-house is entirely stopped for any reason, and the

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water allowed to waste through the overflow at the head of the 14-in. line, a 4-in. relief valve discharging into the tail race affords a safeguard against any ram likely to be produced in the closing of the gates.

The Gate Well.—Fig. 3 gives a very good idea of the general arrangement of the gate well. It is 22 ft. in clear diameter, and is lined with 2 ft. of concrete. In the center of the well is a steel tank 8 ft. in diameter and 21 ft. in height, built of ¼-in. plate. This tank has pipe connections with the reservoir, the distribution and the sewer, and indirectly with the tail race from the Pelton wheels, through which the flow of the pipe line can also be turned independently of the wheels. All surplus water passes through this tank to the waste through the open stand-pipe, the top of which stands at low-water mark for a full reservoir.

From the arrangement of connections and gates, it will be seen that the reservoir or the tank may be temporarily cut out without affecting the supply to the distribution. The overflow is always operative whether the reservoir is in use or not, and the fresh water from the pipe line will always first meet the demand from the distribution, until the supply is exceeded, when the reservoir is drawn upon for the deficiency. A cylindrical brass screen, not shown in the drawing, is inverted over the distribution supply pipe in the tank. This screen can be cleaned at any time by reversing the flow through it. Two feet of concrete is placed in the bottom of the well, enclosing the fittings, and constituting the bottom of the tank. An independent drain pipe provides for the discharge of all seepage water from the well. A spiral iron stairway, supported on the wall by brackets, affords a convenient means of entering the well, while the tank may be entered from above by a stationary iron ladder down the inside. All gates are operated from above by means of hand wheels supported on vertical gas-pipe shafting.

The total cost to the city of the buildings and fittings was $10 177 25.

The Tunnel.

The tunnel through which the distributing system connects with the new reservoir extends directly north from the gate house through the intervening ridge, and is about 1 200 ft. in length. Of this distance, the first 300 ft. from the gate house averages about 20 ft. in depth below the surface, and was tunneled by the contractor in prefer-

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ence to digging open trench. The remaining 900 ft., except for a portion which is through solid rock, is lined with concrete and terminates in a manhole at each end.

Excavation.—The 300 ft. at the south end, which was afterward refilled, was driven through a rather soft argillaceous yellow and blue shale. The remainder, with the exception of about 250 ft. of solid rock near the center of the tunnel, was through an exceedingly hard, compact black shale, which required powder for its removal and rapidly disintegrated when exposed to the air. Timbering was necessary everywhere except in the solid rock section, and consisted of 6 x 6-in. posts and caps, 2 x 12-in. sills and 2-in. lagging. The posts were usually spaced 2 ft. between centers, and were not infrequently crushed in by the swelling of the material, in places where there was apparently no tendency to cave. In places where water penetrated in considerable quantities, the disintegration was so rapid that the tunneling was done with great difficulty, and though timbering was kept close up, several bad caves occurred.

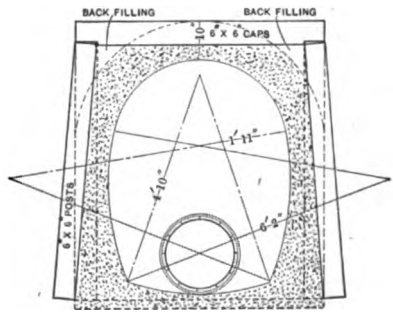

FIG. 4.

The rock was very hard basalt, and was worked entirely by hand. Air was supplied by an exhaust fan operated by an electric motor taking current from the wire of an electric street railway. Driving was usually conducted from both ends. The section driven throughout was 5 ft. 5 ins. in width by 6 ft. in height.

Lining.—It was the original intention to put in a lining of brick with vertical sides and arched top, but the material penetrated proved disappointingly unstable, and the completion of the tunnel driving was so long delayed by reason of the quantity and hardness of the rock encountered, and by the failure of the original contractors, that the work of lining was necessarily performed long after the heavy fall and winter rain had rendered the ground very treacherous, so that it was found inexpedient to remove all the timbers in very many places. It was accordingly determined to use a strong concrete section for the lining, and leave in much of the timbering. Fig. 4 shows the form of section adopted, and the disposition made of the posts in order to

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secure a sufficient thickness of concrete between them and the forms without removing them, the earth being dug away from behind them at the foot of each, and the post pushed back out of the way. The forms were built in 4-ft. sections for the sides, and 2-ft. lengths for the top, and were held in position by 1 x 4-in. bracing on the inside. The concrete was well rammed into all crevices, except at the upper corners, and where considerable caves existed, which were filled back of the concrete with the excavated material. The proportions used in the concrete were the same as those used in the reservoir work, with the exception that the screenings were not removed from the rock. The work of lining in the north end was much delayed by reason of the large amount of digging necessary in securing proper line and grade, this matter having been very carelessly looked after by the men doing the driving. The work in its completed condition is in a very satisfactory shape, which result has been secured only by most untiring supervision.

Cost.—The work passed through many vicissitudes, being at different times under the management of three different contractors and subcontractors. The excavation in the softer materials was not well handled. The rock excavation and concrete work were well managed, and the cost very carefully kept. The cost of the tunnel to the contractors and to the city is given in Table No. 9.

Tunnel Pipe.—The main supply pipe from the new reservoir to a connection with the high-service distribution is 18 ins. in diameter. That in the part of the tunnel which was refilled, and that which lies without the tunnel to the north, is of cast iron $\frac{1}{2}\frac{1}{6}$ in. thick, while the portion lying within the permanent tunnel is wood-stave pipe. Little room was left in the concreted section for the building of pipe, but it was accomplished without difficulty, the gang making an average of 132 ft. a day.

Distributing System.

The distributing system as at present arranged is divided into a low and a high service, the low being supplied from the small reservoir of 500 000 galls. capacity built by the Columbia Water Company at a flow-line elevation of 167 ft.; and the high service being supplied from the new reservoir previously described.

The New Service.—The street mains in the low service range in size from 6 ins. to 14 ins., and in their arrangement and size are designed to

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secure a delivery in case of fire in any of the business portions of the city of 2 000 galls. per minute, in addition to the ordinary domestic consumption, at an effective pressure of 80 lbs. at the hydrant nozzles. Part of the pipe is cast iron, and part steel kalamein pipe.

Astoria has no sea wall, and many of the streets in the business part of the town are supported on pile trestles about 18 ft. high. On these streets the kalamein pipe is used by reason of its lightness and greater adaptability to withstanding vibration than the cast iron. Wherever supporting ground exists, cast-iron pipe is used. The kalamein pipe is supported below the street surface on timbers dapped into and bolted to the piles, and is boxed with 2-in. lumber, the box for the 12-in. pipe being made without a cover. The gauges are the customary standards used by the manufacturers of this class of pipe: 6-in., No. 11½; 8-in., No. 10½; 10-in., No. 10, and 12-in., No. 8.

The hydrants on these streets are connected with the street mains by a section of similar pipe, having a cast flange attached to each end, the pipe being expanded like boiler tubes with a hammer to fit the slightly rounded inner edge of the flange, thus removing any possibility of the hydrant being pushed off by the pressure, while a lead joint provided at the outer end of the flange makes the connection water tight. The cast-iron pipe is all of the hub and spigot type of the following thicknesses:

Diameter, inches	14	12	10	8	6
Thickness, "	¾	11/16	⅝	9/16	½

At the foundry frequent tests of the quality of the metal were made by the inspector with the following results:

		Pounds per square inch.
Test piece, 1 in. square.....	Least strength.....	18 500
	Greatest "	29 500
	Average "	22 000
Test piece, 1⅛ in. round.....	Least "	19 000
	Greatest "	32 000
	Average "	26 500

Test bars 1 × 2 ins., 24 ins. between supports, load applied at center.

	Breaking load.	Deflection.
Least.....	2 000 lbs.	¼ in.
Greatest.....	2 700 "	⅜ "
Average.....	2 230 "	11/32 "

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All pipes were tested to 300 lbs. pressure, and well hammered during the process. Asphaltum was used for coating. The special castings were all made after special drawings, thus dispensing with the unnecessary length and weight of most foundry patterns. Pipes were laid with 3 ft. of earth cover, and with 2 ins. of lead in the joints.

The High Service.—The old pipe taken up from the gravity line of the old works and from the old distribution is largely used for the high service. The former is lap-welded wrought iron supplied with the Converse lock joint, and the latter is principally cast iron of the ordinary hub and spigot type, of a thickness suitable for about 150 ft. head. The pipe for this service ranges from 6 to 10 ins. in size. Be-

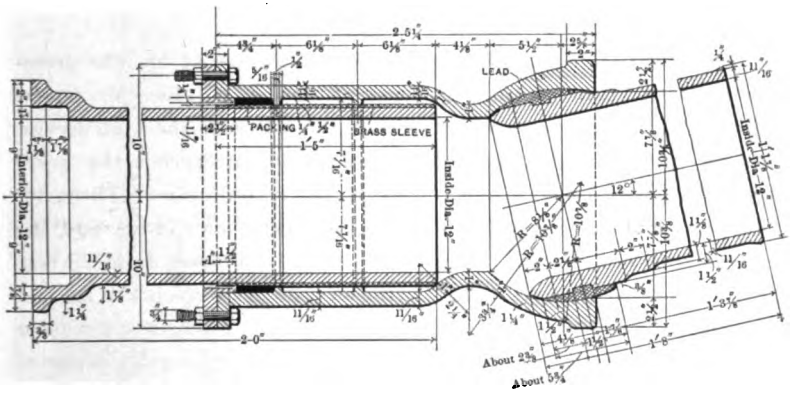

FIG. 5.

fore laying it was all cleaned with steel wire brushes, tested under hydrostatic pressure, and recoated with asphalt.

Hydrants and Gates.—The gates are so distributed as to permit a line to be cut out without disturbing the rest. On the trestled streets, beside being placed at intersections, they are put in every two blocks as a safeguard against loss of pressure by the falling of street mains in event of an extended conflagration. All were tested to 300 lbs. pressure, and are provided with adjustable cast-iron gate boxes. The hydrants have 6-in. connections with street mains, and 5-in. clear valve openings, and are provided with two 2½-in. hoze nozzles, and a 5-in. steamer nozzle. They also were tested to 300 lbs. pressure.

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Flexible Slip Joints.—In some sections of the town the ground is of a very sliding character, the street improvements at some of these points being gradually pushed out of line as much as 2 ft. in a year. In the laying of the street mains, these sections have usually been avoided. It was necessary, however, that about 1 000 ft. of ground of this character be crossed with a 12-in. pipe. To meet the difficulty, a combination slip and flexible joint was devised, shown in Fig. 5. These were distributed at intervals of about 200 ft., and the pipe enclosed for a distance of 50 ft. on each side with a covered wooden box, considerably larger than the pipe, in order to give room for motion, without the pipe having to take the weight and friction of the earth. The ball and socket joint is patterned after the New York City standard, and was so nicely turned that when stood on end, the lead space held water wholly without leakage.

Fire Gates.—Mention has been made that in case of fire, the pressure is secured through the high service from the new reservoir, by the opening of two fire gates which connect the low with the high service at two extreme points, and by automatically cutting out the lower reservoir by means of a check valve in its supply pipe. These fire gates are of the ordinary Ludlow piston lift type, one 14 ins. and the other 10 ins. in diameter, and are both opened and closed from the central fire station by opening and closing an ordinary stop-cock at the end of a line of ordinary ¾-in. service pipe leading to a patented governor attached to each of the gates. This governor was especially designed by the author to meet the conditions here existing, where the lower reservoir elevation was sufficient for ordinary domestic consumption, but insufficient for a really good fire protection directly from the hydrants, while the upper reservoir was at a suitable elevation for a first-class fire pressure, 113 lbs., but objectionably high for domestic use. The device is suited for operating from a distance any piston lift gate working in a pipe under pressure. It would seem to be an economical and efficient arrangement for the speedy closing of gates in large principal supply mains in case of disastrous breaks such as not infrequently occur. It has worked in the present instance without any failure from the beginning, and has proved very certain in action. Its operation is simple, and is readily followed by the aid of Fig. 6. *A* is connected with the pressure pipe or main in which the gate is placed which is to be operated by the governor, *B* with the lower end of

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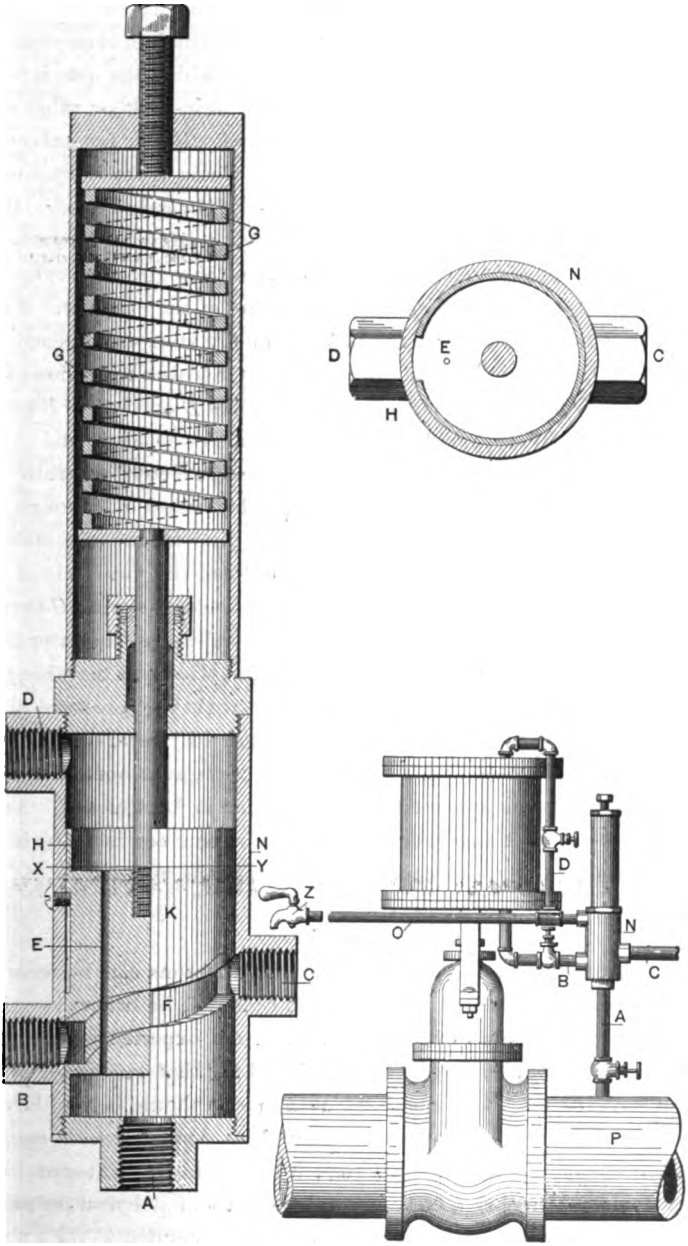

Fig. 6.

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the gate cylinder, *C* with the waste, *D* with the upper end of the cylinder, with a T connection, in one branch of which is connected the small pipe *O* leading to the fire station from which the gate is to be operated. At the end of this pipe is placed a stop-cock or other convenient means of opening and closing this pipe. Under normal conditions the pipe is closed, the water has free access from the pressure pipe *P* through *A*, *E* and *D* to the upper end of the gate cylinder, thus acting on the gate piston and keeping the gate closed. There being an equilibrium of pressure in the two ends of the governor cylinder, the spring *G* keeps the piston at the lower end of the cylinder. When it is desired to open the gate, the pipe at the fire station is opened, releasing the pressure at *D*, causing the governor piston to be thrown to the upper end of the cylinder, which act closes *D* and opens *B* to the passage of the water from *A*. The water acts on the lower end of the gate piston, causing the piston to rise and the gate to open, while the water from the upper end of the gate cylinder passes off through the pipe *O* at the fire station. By closing the pipe at the fire station, equilibrium is again gradually restored between the two ends of the governor cylinder through the opening *E*, when the spring *G* throws the piston to the lower end of the cylinder, releasing the water from the lower end of the gate cylinder through the pipes and movable port *B*, *C* and *F*, leaving the gate free to close with the pressure again restored to the upper end of its lift cylinder. By reversing the connections, it will be seen that a gate can be always kept open instead of closed while the cock at the controlling station is kept closed. As set up, a pair of 1½-in. flanges holding a wire screen are inserted in the pipe *A* to prevent any obstruction getting into the opening *E*.

Conclusion.

In concluding this paper, no instructive lessons can be recorded which are drawn from any failures unanticipated or otherwise, since in every particular the work has been free from experiences of such a nature, and has in all respects accomplished all that was either promised or anticipated. This result the author feels should be very largely attributed to that wisdom which secured as commissioners men of sound judgment, good business sagacity and sterling integrity, and which removed them as a body beyond the pale of political influence. Not the least of their acts of wisdom which contributed to the success

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of the work was their recognition of the responsibility which rested upon the chief engineer, by leaving him entirely untrammeled in the selection of his assistants and the managing of his department. This leads the author to the pleasant task of here recording his sincere gratitude to the engineers associated with him, for their manifestation of high ability, untiring energy, and loyal co-operation in securing the largest measure of success possible. Among those to whom he is much indebted are James D. Schuyler, M. Am. Soc. C. E., who reported on the plans for the work in the fall of 1894, and to whom he is under obligations for many excellent suggestions; R. C. Gemmell, M. Am. Soc. C. E., who, as a partner, was associated with the author in the early preliminary work on plans and specifications; A. S. Riffle, M. Am. Soc. C. E., mechanical assistant on construction, and assistant engineer in charge of reservoir, gate and power house, and tunnel; J. F. Case, assistant engineer in charge of pipe line and head works; George A. Shields, shop and foundry inspector and assistant engineer in charge of the distributing system, and Lars Bergsvik, in charge of construction surveys.

TABLE No. 1.—Cost of Asphalt Coat on Slope Concrete—29 637 Sq. Ft.

Labor.	Total.		Total cost.	Total cost per square foot.
	Hours.	Rate.		
Buildings, sheds, etc.	.....	.....	$5.00	$0.000168
Spreading	91	At $0.20	18.20	0.000614
Boiling	91.5	" 0.15	13.72	0.000462
Attendance	73.5	" 0.15	11.025	0.000372
Sweeping	49.5	" 0.15	7.425	0.000251
	305.5			
Material.	**Quantities.**			
Asphalt	19 243 lbs.	" 0.01225	235.729	0.007950
Haul	9.6 tons.	" 0.4700	4.500	0.000152
Fuel	1.0 cord.	" 2.50	2.500	0.000084
Totals	.....	.....	$298.102	$0.010063
Contract price	.....	.....	$657.94	$0.022222

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TABLE No. 2.—Cost of First Coat of Asphalt on Concrete Bottom—34 454 Sq. Ft.

Labor.	Total.		Total cost.	Total cost per square foot.
	Hours.	Rate.		
Building sheds	25	At $0.20	$5 00	$0.000146
Spreading	38	" 0.20	7 60	0.000220
Boiling	37	" 0.15	5 55	0.000161
Attendance	43	" 0.15	6 45	0.000186
Sweeping	44	" 0.15	6 60	0.000191
	162			
Material.	**Quantities.**			
Asphalt	18 490 lbs.	" 0.01223	226 50	0.006578
Haul	9.25 tons.	" 0.470	4 35	0.000126
Fuel	1.0 cord.	" 2.50	2 50	0.000073
Total cost			$264 55	$0.007681
Contractor's price			$765 58	$0.0222222

TABLE No. 3.—Cost of Second Coat of Asphalt on Bottom—34 154 Sq. Ft.

Labor.	Total.		Total cost.	Total cost per square foot.
	Hours.	Rate.		
Building sheds			$5 00	$0.000146
Spreading	35	At $0.15	5 25	0.000153
Boiling	30	" 0.15	4 50	0.000132
Attendance	52.5	" 0.15	7 88	0.000230
Sweeping	44.5	" 0.15	6 68	0.000195
Foreman	17.5	" 0.25	4 38	0.000128
Material.	**Quantities.**			
Asphalt	19 591 lbs.	" 0.01225	239 99	0.007019
Haul	9.8 tons.	" 0.470	4 61	0.000134
Fuel	1.0 cord.	" 2.50	2 50	0.000073
Total cost			$280 79	$0.008210
Contractor's price			$753 90	$0.022222

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TABLE No. 4.—Cost of Laying on Slope Brick Dipped in Asphalt —132 000 Brick, 29 637 Sq. Ft.

Labor.	Total. Hours.	Total. Rate.	Total cost.	Total cost per "M."
Unloading brick from barge	290	At $0.15	$49 00	$0.37122
At wharf	22	" 0.25		
Haul and storage at the Reservoir	160	" 0.35	152 43	1.15473
	140	" 0.55		
	129.5	" 0.15		
Laying	561	" 0.15	84 15	0.63750
Attendance	1 341	" 0.15	201 15	1.52337
Briding asphalt	220	" 0.15	33 00	0.24999
Foremen	96	" 0.25	24 00	0.18180
	2 959.5			
Material.	**Quantities.**			
Brick at	.....	" 7.00	924 00	7.00000
Asphalt	93 372 lbs.	" 0.1225	1 143 81	8.66516
" haul	46.7 tons.	" 0.47	21 95	0.16628
Total cost	.....	.....	$2 633 49	$19.95055
Contract price	57 000	.....	$2 500 50	$21 50
	75 000			17 00

TABLE No. 5.—Cost of Asphalt Finishing Coat on Brick Slope Lining, 29 637 Sq. Ft.

Labor.	Total. Hours.	Total. Rate.	Total cost.	Total cost per square foot.
Building sheds, etc	.....	.....	$5 00	$0.000168
Spreading	95.75	At $0.15	14 36	0.000485
Boiling	73.25	" 0.15	10 99	0.000371
Attendance	144.50	" 0.15	21 68	0.000732
Sweeping	20	" 0.15	3 00	0.000101
Foreman	60	" 0.25	15 00	0.000505
	393.5			
Material.	**Quantities.**			
Asphalt	25 230 lbs.	" 0.01225	309 07	0.010425
Haul	12.6 tons.	" 0.47	5 92	0.000199
Fuel	1.0 cord.	" 2.50	2 50	0.000084
Totals	.....	.....	$387 52	$0.013070
Contract price	.....	.....	$657 94	$0.022222

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TABLE No. 6.—Cost of Filling with Mastic Crevices in Finished Brick Slope, 29 637 Sq. Ft.

Labor.	Total.		Total cost.	Total cost per square foot.
	Hours.	Rate.		
	146	At $0.15	$21 90	$0.000738
	130	" 0.15	19 50	0.000657
Attendance and sweeping	56	" 0.15	8 40	0.000285
Foreman	30	" 0.25	7 50	0.000250
	362			
Material.	**Quantities.**			
Asphalt	2 813 lbs.	" 0.01225	34 46	0.001164
" haul	1.4 tons.	" 0.47	0 66	0.000022
Fuel	1.0 cord.	" 2.50	2 50	0.000084
Total cost			$87 92	$0.003

Contractor's price included with coating.

TABLE No. 7.—Cost of Ironing Slope, 29 637 Sq. Ft.

Labor.	Total.		Total cost.	Total cost per square foot.
	Hours.	Rate.		
Ironers	29.55	At $0.15	$44 33	$0.001496
Heaters	75	" 0.15	11 25	0.000380
Sweeping and attendance	34.5	" 0.15	5 18	0.000174
Foreman	49.5	" 0.25	12 37	0.000418
	454.5			
Material.	**Quantities.**			
Irons	20	" 1.50	30 00	0.001012
Fuel	1.0 cord	" 2.50	2 50	0.000084
Total cost			$105 63	$0.003564

Contractor's price included with coating.

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TABLE No. 8.—Cost of the Reservoir. Capacity, 6 203 000 Galls.

Items.	Amount.	Total contract price.	Total cost to contractor.	Additional cost to city.	Total actual cost.
Excavation	49 540 cu. yds.	$7 431 00	$8 371 99	.....	$8 371 99
Concrete on slope	603.3 " "	2 111 55	1 816 19	$1 702 75	3 518 94
" " bottom	678.2 " "	2 373 70	1 761 83	1 790 95	3 552 78
" in pedestal	18.0 " "	63 00	69 11	44 10	113 21
" " tunnel	12 6 " "	44 10	34 41	30 87	65 28
Cement coating, bottom	341.54 squares	.....	129 63	254 80	384 43
" " pedestal	3.00 "	.....	4 07	2 45	6 52
1st coat asphalt slope	29 637.0 sq. ft.	657 94	298 10	.....	298 10
2d " " "	29 637.0 " "	657 94	387 52	.....	387 52
1st " " bottom	34 454.0 " "	765 58	264 55	.....	264 55
2d " " "	34 154.0 " "	758 90	280 79	.....	280 79
Brick lining	132 0 M	2 500 50	2 633 49	.....	2 633 49
Mastic on brick lining	29 637.0 sq. ft.	.....	87 92	.....	87 92
Ironing slope	29 637.0 " "	.....	105 63	.....	105 63
Parapet wall	289.0 cu. yds.	1 445 00	1 304 73	475 58	1 780 31
Iron fence	917.2 lin. ft.	.....	.....	985 40	985 40
Riprapping bank	.....	.....	.....	36 23	36 23
Fixing road	.....	.....	.....	17 40	17 40
Cleaning up	.....	.....	59 82	.....	59 82
Grading around reservoir	800.0 cu. yds.	120 00	85 00	.....	85 00
Office buildings, etc.	.....	.....	64 18	.....	64 18
Water supply and main	.....	.....	288 00	.....	288 00
Total	.....	$18 929 21	$18 046 96	$5 340 53	$23 387 49

TABLE No. 9.—Cost of Tunnel.

Items.	Quantities.	Total cost to contractors.	Total contract price.	Additional cost to city.
Earth excavation	963.5 cu. yds.	$1 795 43	$963 50	
Timbering	.....	587 38	887 00	
Earth excavation, north approach	157.0 cu. yds.	37 00	157 00	
Earth excavation, south approach	.....	120 57		
Cleaning and relagging	.....	16 50		
Rock excavation	296.7 cu. yds.	3 290 68	519 23	
Concrete lining	563.5 "	2 252 27	3 381 00	$1 161 30
Total cost	.....	$8 039 83	$5 857 73	$1 161 30

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DISCUSSION.

Mr. Hering. RUDOLPH HERING, M. Am. Soc. C. E.—A commendable feature of the work described in the paper is the high degree of thoroughness that was not only proposed but apparently attained in all the important details. It is also to be commended that an accurate test was made of the leakage of the reservoir and the carrying capacity of the pipe line before they had been brought into use, which, unfortunately, is not always done. Owing to these early measurements, it will be a simple manner to observe any gradual change in the capacity of the pipe and to indicate the true causes for it.

It is interesting to see the comparative results obtained in the flow through wooden stave and steel pipes. The Kutter coefficient found from the former by experiment, namely, $n = 0.00985$, is less than that obtained by the Darcy-Bazin formula in a semi-circular channel of planed boards nearly 5 ft. in diameter, which is about $n = 0.0118$; but it equals almost exactly their result obtained in a small rectangular channel, 4 ins. wide, where it is $n = 0.0097$. When a high degree of smoothness exists in small channels or pipes, the coefficient varies much more rapidly than in rough channels; that is, a very slight change in the character of the surface makes a material difference in the flow. It is therefore especially important in such cases to have an actual measurement if possible, and not to depend solely upon formulas for very close results.

The steel plates were apparently made of sheets 4 ft. in length. These plates were No. 10 and No. 12, B. W. G., and therefore 0.134 and 0.109 in. in thickness, and were riveted with lapped joints in alternating large and small courses, the latter having the full nominal diameter of the pipe. Every eighth course was made slightly conical. Straight seams were double riveted, and round seams single riveted. For this pipe the Kutter coefficient was found to be $n = 0.0111$. Hamilton Smith, Jr., M. Am. Soc. C. E., obtained for riveted pipe of similar diameter and thickness of plate $n = 0.010$ to 0.011. The greater friction in the steel pipe than in the wooden stave pipe is undoubtedly due to the projecting plates at the joints and to the rivet heads. The smoothness of the surface of the two materials does not differ much.

The importance of the projections due to the thickness of the plates, as affecting the resistance, and as represented by the Kutter coefficient n, is again shown by this new gauging. Here the projections are 0.134 and 0.109 in., and n is found to be 0.0111. In the East Jersey Water Company's pipe, about 4 ft. in diameter, the projections are 0.25 to 0.375 in., and n averages 0.0148. In the wrought-iron riveted flume at Holyoke, Mass., about 8½ ft. in diameter, the projections are 0.36 in., and n equals 0.016.

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Mr. Hering. The speaker was glad to see the continued use of stave pipe advocated, because it was certainly, at least for the Pacific coast, an excellent construction. It seems to outlast steel and iron, from the fact that the pressure of water from the interior causes the pipe to be continually saturated, and this seems to preserve it from rot, which is not the case with wood under other conditions. For that reason it has been suggested on the Pacific coast not to coat the pipe either inside or outside. It has been found there that if the stave pipes were coated on either or both sides they were more apt to rot than if they were left in a natural condition, because the coating would prevent the water from penetrating the wood as freely as necessary. It was assumed in a case that the speaker was familiar with, that such uncoated stave pipes would last at least twenty-five or thirty years, but no actual experience has, of course, been had with it for that length of time.

In referring to the asphalt lining of the reservoir, the speaker approved of the author's reasoning, having been careful to base his recommendations on the very latest results arrived at on the Pacific coast, where a number of asphalt-lined reservoirs of many different constructions have been built. For the slopes, where the trouble usually occurs, the author's recommendation was probably the best for the purpose, and one likely to give satisfaction also in the East, namely, first giving a coating of asphalt, then placing a layer of bricks laid flat upon it after they have been dipped in asphalt, and finally putting on another coating to fill up the joints. A very thin coat of asphalt or asphalt-concrete also appears to be satisfactory in similar cases. The brick lining was recently adopted as the best lining for the slopes of the Queen Lane Reservoir in Philadelphia.

CORRESPONDENCE.

Mr. Weston. EDMUND B. WESTON, M. Am. Soc. C. E.—As the writer has advocated the use of Darcy's formula,* he is pleased to see the close agreement between the experimental results given in the paper for the 16-in. steel pipe, and results that he has computed by Darcy's formula. The latter would give, considering the discharge and length given by the author (6.369 and 1 000), a loss of head due to friction of 5.1291, instead of the experimental result 5.0023, or about 2.5% more; and taking into account the friction head and length given by the author (5.0023 and 1 000), the formula would give a discharge of 6.2898 instead of the experimental result 6.369, or about 1.25% less. As will be seen by these figures, there is but a very slight difference between the results obtained by Darcy's formula and the experimental results, and the variation is upon the safe side.

* *Transactions*, Vol. xxii, p. 1.

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Mr. Russell. S. BENT RUSSELL, M. Am. Soc. C. E.—The kind of reservoir lining used at Astoria may properly be compared with the lining used for the bottom of the new settling basins at St. Louis. There are six of these basins each 670 ft. by 400 ft., and from 11½ to 18 ft. in depth of water when full. They have been in continuous use for over a year. The floor of these basins consists of 18 ins. of clay puddle protected by 6 ins. of concrete, which was laid in blocks 7 ft. square, with a ½-in. joint. These joints were filled with a composition of equal parts of No. 6 paving pitch and powdered carbonate of lime. This composition becomes quite soft under the summer sun, but is always somewhat hard when submerged.

The basins are surrounded by heavy masonry walls, which rest on compressible earth. Since completion the walls have settled considerably, several inches in some places. This sinking of the walls seems to result in distortion of the adjoining bottom, as the blocks nearest the walls are found to have settled, and in some places they are cracked. Where the floor block has settled much, the pitch joint 7 ft. from the wall is found to have opened. Away from the walls, and usually adjoining walls that have not settled, the floor lining has proved a success so far. The joint filling rises when the blocks are warm and goes back when they are cool, as described by the author. The floor lining seems to have adjusted itself perfectly to all ordinary inequalities of settlement.

These basins will probably be repaired by taking up the floor blocks that have settled badly, and replacing them with blocks 7 ft. by 2 ft. and 8 ins. deep, laid with their major axis parallel to the wall. The joints will be filled with a somewhat stiffer composition, probably equal parts of No. 6 paving pitch, carbonate of lime, and clean, dry river sand. It is thought that the narrower blocks and greater number of joints will provide for all further settlement that may take place.

In operation these basins are drained and cleaned four or five times a year, and the bottom is thus exposed to more severe treatment than is the bottom of an ordinary service reservoir.

From this experience, the writer is inclined to fear that floor blocks 20 ft. square, as described in the paper, might prove a failure on ground at all unstable. It seems questionable, too, whether the coating of asphalt on top of the blocks would be much protection in case of broken blocks or distorted joints. As the coating adheres to the top of the floor block, the strain on the coating would be concentrated over the joint or crack, and the continuity of the coating would be greatly endangered. These suggestions are based on the assumption that the pitch composition and asphalt act in much the same way, as would seem probable from the nature of the two materials.

Mr. Harlow. JAMES H. HARLOW, M. Am. Soc. C. E.—The writer's experience leads him to doubt the correctness of the views expressed in the following quotation from the paper:

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"The author holds in the much-discussed question to which this gives rise, that though a private person or company may often with creditable discretion discard a bid that is too low, yet in handling the funds of a municipality, if the proposal is strictly formal, and the securities satisfactory, the administrator of such funds cannot, with due propriety and proper regard for the wishes of the public whom it serves, discard such a bid." Mr. Harlow.

The writer cannot see how it is more creditable for a municipality to obtain something for nothing, or something for less than its value, than for a private individual. The contractor may make a mistake in his bid for any given work, but when this mistake is self-evident, such bids should be rejected. An engineer can be more lenient in the enforcement of the detail specifications when the contractor is making money. If the contractor is losing money, he urges with a good deal of force that leniency granted on one point should be granted on other points where the engineer should carry out the exact wording of the specifications.

An example is afforded by the case of a borough in the vicinity of Pittsburg, where about ten years ago the municipality decided to construct water-works. The borough's engineer prepared the necessary specifications and plans, and bids were asked for. The proposition of the lowest bidder was from a party who had considerable previous experience in the construction of railroad embankments and culverts. He was a rather ignorant man, with no experience on work similar to that on which he had now bid. The second lowest bidder was a man with much experience in the construction of water-works, and whose bid was about $4 000 above that of the lowest bidder. As to the financial responsibility, the second party stood better. Both parties were able to give suitable bonds.

The engineer reported against awarding the contract to the lowest bidder, his argument being that the bid was entirely too low. The Borough Council decided to award it to the lowest bidder, believing that in case of loss it would not fall upon the borough, for it had sufficient bonds to compel the construction of the works as designed. After the award of the contract, and after the contractor had been working some weeks, the work began to drag. Complaints were heard that the work was not properly managed, and the laborers were not being paid the full amounts due them; and soon it became necessary to forfeit the contract, and the bondsmen were called upon to complete the work. This they did. The report is, that the bondsmen divided up the cost of several thousand dollars between them. So far as the borough was concerned, the work was completed, and to the satisfaction of the engineer and other officials.

Several of the bondsmen lived in this borough. One of them was crippled in his business by the withdrawal of his share of the expense on account of the contract; the result was that he failed. Accord-

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Mr. Harlow. ing to newspaper reports, labor bills to the amount of some $1 500 were outstanding, and attempts were made to collect from the borough, and, finding this could not be done, an attempt was then made on the bondsmen. The writer does not know exactly how this point was settled, but his impression is that these laborers never received their money, the bondsmen claiming that while they were responsible to the borough for the satisfactory completion of the work, they were not responsible to the laborers or for material furnished the contractor prior to their assumption of the work. A number of these laborers were residents of the borough.

Some two or three years after this, the writer was called upon to design a water supply system for the adjacent borough, and during his investigations there became acquainted with a number of men who had furnished material to the contractor mentioned previously. These claims had never been settled. They varied from $800 for cement down to $75 and $50 for sand and other materials. The aggregate of the bills that the writer found was something in the neighborhood of $2 500. These material men resided in the borough.

It has always been a question to the writer whether the borough was justified in the awarding of this contract to the party it did, remembering that the borough is made up of individuals, and a number of these individuals had to pay the difference in the cost.

The author says that "by a combination of circumstances, fortunate to the city, the works have been completed at a remarkably small cost." From what precedes this item, has not a considerable difference been paid, not by the city as a city, but by individual and innocent inhabitants of the city? This is apparently the fact, for the reason that the city apparently paid something over $4 500 in excess of the contract price for the reservoir.

Mr. Gould. E. SHERMAN GOULD, M. Am. Soc. C. E.—The gaugings on page 4 show a steady diminution in the minimum flow during the four years of observations. In 1893 the flow above the mouth of Cedar Creek was 2 646 000 galls. per 24 hours, and in Cedar Creek 718 000 galls. per 24 hours. These two flows combined, as the writer understands it, would be the amount furnished by the stream at the point of diversion, and would aggregate 3 364 000 galls. per 24 hours. This is not an encouraging outlook for a supply of 4 000 000 galls. per 24 hours, with a storage of only 50 000 000 galls., or only enough for about 12 days' consumption.

All that is said about the wooden pipe is of great importance. This kind of conduit can evidently be used only under light pressures, say 100 lbs. per square inch as a maximum. This materially limits its availability. It is to be regretted that more details, with a longitudinal section through the joints, were not given of the steel pipe, which seems to be of a somewhat unusual type.

The increased flow in the wooden pipes was to be looked for, owing

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to the greater regularity of its interior surface. Had a steel pipe been used in its place, with the given grade, a diameter of about 19 ins. would have been required, so the gain is only about 5% in favor of the wood. A query naturally arises as to the diameter of the wooden pipe being strictly 18 ins. It must be difficult to ascertain the exact working diameter of such a pipe. The hydraulic grade line of the steel pipe seems to have been less satisfactorily determined than that of the wooden one, for, on account of its greater height above the pipe line, no stand pipes were placed upon it. Taking the data as given by the author, the flow is large for a pipe of that character and diameter. Here, too, there must be some difficulty as to true diameter, because, for one thing, the longitudinal lap joint in so small a pipe is a confusing element. Mr. Gould.

The proportions of the concrete seem to have been in round numbers, 1 cement, 0.8 sand, 4 gravel and 7 broken stone. This seems a rather lean mixture for hydraulic work, but the cement being good Portland, there is no doubt that, with careful mixing and placing, satisfactory results could be confidently anticipated. It is stated that 1 558 barrels were used to make 1 282 cu. yds. of concrete. This makes, including plastering and waste, nearly 1¼ barrels per cubic yard, which represents good, honest work under the above proportions. The quantity put in per day on the bottom, 2.35 cu. yds. per man, if by per man "all told" is meant, is a very high average and represents smart work.

Were the joints really "expansion" joints; that is, did the asphalt filling them compress and dilate after the fashion of india-rubber? If the underlying concrete contracted and expanded, what effect did this movement have upon the sheet of asphalt spread over the surface? To the writer's mind, one of the great advantages of concrete consists of its being used in a continuous mass. When building the Palatino Reservoir at Havana, Cuba, a few years ago, he had occasion to use concrete as a bottom lining upon a fairly large scale. This was a double reservoir, the bottom surface of each compartment being of about 60 000 sq. ft. The thickness of the layer of concrete was about 1 ft. Imported Portland cement was used, the proportion being one part of cement, three of sand, and six of broken stone. The sheet was spread continuously without joints, and never showed signs of any material movement. The work was of the best character, and the finished concrete kept constantly moist for a long time—perhaps a week or two, or even longer—after being laid. Constant wetting of all masonry was insisted upon. After a month or two the concrete resisted the action of a sharp pick. It is true that the range of temperature to which it was exposed was probably much smaller than at Astoria, but the result was so perfect in every respect that the writer would need strong proof to convince him of the expediency of foregoing the great advantage of keeping his floor absolutely continuous.

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Mr. Riffle. FRANKLIN RIFFLE, M. Am. Soc. C. E.—In defending the action of the Water Commission in awarding the contract for divisions 1, 2, 3 and 7 to the lowest bidder, the author makes the following explanatory statements:

First.—The proposal was "formal in the last degree."

Second.—The contractor was "determined to have his rights recognized at any cost."

Third.—Although "a private person or company may often, with creditable discretion, discard a bid that is too low, the administrator of public funds cannot with due propriety and proper regard for the wishes of the public * * * discard such a bid."

The first two statements convey the impression that the contractor's "rights" called so loudly for recognition that the Commission was legally powerless to exercise any discretion in the matter. The writer, being acquainted with the facts, is of the opinion that the contractor had no rights to enforce, either legal or moral. He had become hopelessly involved in a paving contract in a neighboring city only a few months before, and deemed it expedient, prior to submitting a proposal at Astoria, to incorporate under a new name. The engineer had, with commendable foresight, introduced into his specifications the following provisos, which are singularly applicable to the case in hand:

"The Water Commission reserves the right to reject any or all bids, or parts of bids, which they may not deem advantageous to the city."

"No proposal will be accepted from any firm or person known either to the Commission or the engineer to be in arrears on other contracts, or to have failed in other contracts to comply with the specifications, and fulfill his contracts."

Just why these excellent stipulations were waived by the Commission is not apparent, unless the explanation be found in the author's last statement. Here he virtually takes the untenable position that conservative business methods cannot with propriety be employed in the administration of public affairs. He assumes that such a procedure is contrary to "the wishes of the public." The defence becomes weaker still when it is considered that the Astoria Water Commission was composed of representative business men, and was clothed by the State legislature with large discretionary powers. The practice of indiscriminately awarding contracts to the lowest bidder promotes incompetency and rascality. In the case under discussion the contractor, having nothing to lose, could well afford to take chances which a responsible contractor dare not take. So long as he could satisfy his creditors with partial payments he would be able to appropriate a good portion of each monthly payment received from the city to his individual use. This was actually done. In the end he was better off financially than if he had failed to secure the contract.

It is to the engineer, rather than the average municipal board or commission, that one would naturally look to advocate the rejection

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of undesirable bids. He is presumed to know the difficulty of obtaining good results when the contractor is losing money, to say nothing of the probability of vexatious and expensive complications, which may entail a serious loss upon the city. The writer holds, therefore, that it should be the duty of the engineer of either a public or private enterprise to determine which one of the several bids received is the lowest and the best, and to urge its acceptance without fear or favor. Mr. Riffle.

The location of the pipe line was evidently made with great care, and the result certainly justifies the painstaking methods employed. In a country such as that described by the author a carefully surveyed preliminary line, supplemented by an accurate contour map, is almost an absolute necessity. Without it an intelligent, economical location is practically impossible.

The expense, however large, is fully justified. The advantage of a contour map as an aid to location is clearly pointed out in the author's statement that the length of the line as finally located is one mile shorter than the preliminary line, a saving in distance of about 10 per cent.

The location of the Astoria reservoir is known to the writer as having been made only after considerable investigation. At Astoria the hillsides in many places consist of unstable material, which has a tendency to slide when saturated with water during the rainy season. Excavations disturb the equilibrium of these hillsides to such an extent that the most serious consequences often ensue. The engineers who had charge of the construction of the Oregon Railway and Navigation Company's lines, and the Cascade Division of the Northern Pacific Railway were given much trouble by slides, and although these roads have been in operation for a number of years, each successive rainy season brings a recurrence of the evil. This feature of the hillsides of this section of the Pacific Coast is so thoroughly understood by most of its residents, that the construction of two of Portland's large reservoirs in a ravine, one side of which was clearly unstable, occasioned considerable comment. As was reasonably predicted the hillside began to slide before the completion of the work, and although eighteen months have elapsed since their completion, the reservoirs have been undergoing repairs during this period at a large expense. It is but fair to add that, although the construction was under the charge of one of the most prominent engineers of the Pacific Coast, he was in no way responsible for the selection of the sites.

The Astoria reservoir site was selected by the author about the same time as the Portland reservoirs were located. His attention being called soon after to the sliding character of the ground in the immediate vicinity, he began a thorough investigation which ended in the rejection of his first choice, and in the selection of a site in another portion of the city where there was no question as to stability. The

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Mr. Riffle. author stated to the writer that in making the change he was guided by his own judgment, supplemented by the opinions of local men of intelligence who knew much more about the peculiarities of the country from long residence than he did. Such a spirit is highly commendable, and stands out in sharp contrast with the practice of engineers who consider it beneath their professional dignity to make use of knowledge obtained from what they choose to term humble sources.

The writer does not believe that the publication of the cost of work in detail, as has been attempted by the author, is attended with beneficial results. While engineers of experience fully appreciate the fact that local conditions must be taken into consideration in applying the cost figures obtained in any locality to another locality, incompetent and inexperienced engineers are very apt to be misled by such figures. It must also be borne in mind that the bare cost of labor is but one item in the cost of work to the contractor, and although it is usually the largest item, it often falls far short of the total cost. Office and traveling expenses, freight charges, depreciation of plant, etc., are items that oftentimes assume large proportions, and are strictly chargeable to the cost of the work. Without these items, tables such as those included in the paper under discussion, are misleading for the purpose of estimating the cost of work. While an experienced engineer would undoubtedly add a liberal percentage for items not included in the tables, an incompetent engineer would unhesitatingly appropriate them verbatim.

Mr. Henny. DAVID C. HENNY, M. Am. Soc. C. E.—In describing the details of the Astoria stave pipe, the author calls attention to the difficulty experienced in obtaining fir staves of a quality suitable for pipe construction. Douglas fir is a very superior wood in respect to strength, stiffness and straightness of grain, and has the further advantage of being cheap and readily obtainable in large quantities free from knots. Yet it possesses qualities which render it less desirable for use in stave pipe than would at first appear. It is subject to considerable shrinkage and checking, which is of importance when finished staves are to be shipped and hauled long distances and left exposed some time along the pipe trench. It was a fortunate circumstance in Astoria that the pipe line runs for its entire length through the thick woods, where the staves were protected from the sun and wind.

Another disadvantage of fir is that its grain is not so close as that of other woods, such as cypress and redwood, necessitating great care in the selection of the staves used in heavy pressure pipe, to prevent percolation. The principal disadvantage is, however, that pitch seams are so common that a certain percentage is permitted, even in the ordinary clear grade as occurring in the market, and that to obtain fir absolutely free from pitch seams requires a weeding-out process from beginning to end, very discouraging to the proprietor of the mill, and

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most difficult to carry through without compromise. A pitch seam is caused by the accumulation of pitch between two annular rings; therefore it is not necessarily objectionable in a bastard grain stave, unless too large. In a quarter-sawed stave a pitch seam showing on both sides is sure to cause a leak. If it shows on one side only, and penetrates but little, no leakage would result from it; but the depth of penetration cannot be definitely ascertained, except by sawing the piece in two. Of two pitch seams, similar in appearance and permitting the insertion of a knife point to the same depth, one will often be found to be shallow and the other to go nearly through. Mr. Henny.

Rigid requirements and inspection of the wood necessarily lead to a higher price of the pipe, to which there is, with some engineers, a dangerous tendency to object, seemingly from a desire to make the saving resulting from the use of stave pipe unduly large.

The specifications under which the Astoria wooden pipe was built prescribe in detail the character of all materials to be used, as well as their quantities and dimensions, and provide that all work shall be done in a manner satisfactory to the engineer. Finally there occurs in the specifications the following clause:

"In the erection and filling of pipe great care must be exercised by the contractor to prevent overstraining of the bands. The final test of the satisfactory fulfillment of the above requirements shall be that no band on being removed from the pipe, after the latter has been subjected to its natural working pressure for at least thirty consecutive days, shall show any appreciable fixed increase in its original length; and the failure of 5% of the bands examined to meet this requirement, shall be deemed sufficient grounds for the rejection of the entire work, until such defects have been entirely repaired in a manner satisfactory to the engineer."

It will be observed from this, that after the contractor shall have furnished the materials and performed the work in all respects as required by the engineer, he is still liable to have his entire work rejected although the pipe line should prove to be entirely tight.

The experiments mentioned by the author on page 18 were undertaken to determine the extent of the risk involved. The results were satisfactory, in that they indicate that the highest strain that could be produced by continued cinching of the band on the kiln-dried section of pipe is well within the elastic limit of the steel, and that there is a tendency for this strain to drop back speedily to the safe limit of about one-fourth the ultimate strength of the bands. This is due rather to a yielding of the fiber under the round band than to lack of compressive strength of the wood in the direction of its width. It should be understood that a wooden pipe, when properly built, has its bands cinched and hammered into the wood until a width of bearing is procured about equal to the radius of the band, which may be done without injuring the fiber.

A lineal inch of a ½-in. band, as was used in the experiment, has a

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Mr. Henny. bearing on the wood of about ¼ sq. in. A strain on the band of 4 750 lbs. indicates a pressure per lineal inch of band on the wood of 458 lbs., or about 1 832 lbs. per square inch of bearing, which, as might have been expected, could not be permanently sustained by water-soaked fir, regardless of the simultaneous tendency of the wood to swell, and this sufficiently accounts for the gradual reduction of the strain as recorded.

The limit of compressive strength of the staves in the direction of their width was not reached in any of the experiments mentioned; and hence the author's conclusions on this point and on the consequent limit of safe pressure in wooden pipe construction are believed to be very conservative.

The author states that yellow fir is a hard and rather unyielding material. This statement is probably intended to apply to the wood when perfectly dry, for the writer has found that fir, when partly green, can be readily compressed, as was proved in the cinching of the Astoria pipe. The cinching is intended to be carried on to a point where the width of the stave is forcibly reduced to a lesser width than would have resulted from shrinkage in drying, and this could be accomplished where the banding was close. Where the bands are spaced far apart, continued indentation of the band rather than further reduction of the width of the stave results. Light-banded pipe, if it is to be tight upon first filling, should be back-filled immediately after cinching, or, if not to be covered, should be recinched just before filling.

The author states that the wooden pipe at Astoria was entirely tight from the beginning, and that this is contrary to the usual experience with stave pipe. While such absolute tightness at first filling as was here attained may be exceptional, the writer believes it to be entirely within the bounds of skilful workmanship and inspection to avoid from the start practically all causes of leakage.

The author's measurements of the actual amount of leakage of the Astoria conduit are very instructive, especially as they illustrate the comparative tightness of the wooden and the steel pipe in this instance. They are of the more value because the opportunity of taking accurate measurements of leakage of pipe lines is usually either lacking or allowed to remain unimproved.

In determining upon a diameter of 18 ins. for wooden pipe, the author had in view the advantage of rendering the inside of the pipe accessible for purposes of construction and inspection. As final inspection is done from the outside, the main advantage lies in the ease of rounding the pipe out from the inside. In this connection it may be stated that in his recent practice the writer has used, on pipe smaller than 18 ins., rounding-out machines, of simple construction, inserted in the pipe and operated from its open end, which perform their work as perfectly as can be done by an inside man with a mallet. While

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the constructional advantage is the only reason advanced for the difference in the diameters of the wooden and the steel pipe, it appears that the author has incidentally put into practice the economical principle in the design of long pipe lines with greatly varying pressures, which demands the saving of available head on the cheaper or light pressure portions of the line and its concentration upon the more expensive or heavy pressure portions, and it is believed that an actual saving has resulted in this case from the adoption of a broken grade line. Mr. Henny.

The author's experiments on frictional loss are a welcome and needed addition to engineering knowledge, valuable especially on account of the accuracy with which both the hydraulic grade line and the flow have been ascertained. The close agreement between the assumed and the measured hydraulic grade lines is remarkable. The coefficient of roughness of 0.01 usually assumed for wooden stave pipe is shown in this case to have been slightly in error on the safe side. It may be appropriate to state here that on the wooden pipe line at Astoria there are, in addition to a succession of sweeping horizontal and vertical curves, twenty-seven cast-iron bends, with a radius of curvature of 5 ft., and with an average central angle of about 31°, conditions more unfavorable than are usually met with.

The reservoir at Astoria is located on the top of a knoll on the side of a range of hills away from the Columbia River and the city. If this location was selected to guard against danger of landslides, which, owing to the geological formation in connection with the copious rainfall during the winter season, are rather common in the northwestern portion of Oregon, the extra expense of a tunnel, which is necessitated, was most wisely incurred. Experience with two new reservoirs in a neighboring city, completed since the location of the Astoria reservoir was decided upon, indicates that neither extreme care in construction nor the exercise of every practicable precaution in strengthening the lining or sub-draining the slopes is adequate to prevent failure of a reservoir built in sliding ground.

It has become rather customary on the Pacific Coast to make water power development an incidental part of work in connection with gravity pipe lines originally intended for domestic supply only. The writer questions the wisdom of this, in some cases, and believes that the difference in cost of the work with and without this addition is not always ascertained, and a comparison of the resulting cost of water power with that of steam power is often deemed superfluous. He mentions this, not because of a doubt that such comparison has been made in this instance and the extra outlay has seemed justifiable, but because comparative figures would be interesting, especially considering the cheapness of fuel in Astoria.

The detailed statements of the actual cost of portions of the work add materially to the value of the paper. As they are based, however, on the reports of timekeepers, and in the nature of the case cannot

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Mr. Henny. take into account the many miscellaneous and unavoidable expenses of the contractor, which at times exceed the percentage ordinarily estimated by the engineer for contractor's profit, they necessarily understate the real cost of the work by an unknown amount, and if made the basis for future estimates it is evident that their use cannot dispense with experience and good judgment.

Mr. Adams. ARTHUR L. ADAMS, M. Am. Soc. C. E.—The author is pleased that the action of expansion joints in concrete construction, as described in connection with the Astoria reservoir, has been verified by Mr. Russell in his observation of somewhat similar joints in the St. Louis settling basins, especially since Mr. Gould seriously questions that the joints act as described. The purpose of the joints in the reservoir bottom was to prevent temperature cracking rather than to anticipate any considerable settlement, and for this purpose a joint every 20 ft. seems sufficient. The thin coating of asphalt applied to the finished bottom concrete is intended as an additional safeguard against cracking from a similar cause, besides rendering the bottom less pervious to water. Such a coating cannot be expected to maintain its continuity in case of very serious breaking up of the underlying concrete. A course of brick dipped in asphalt and laid on top of the concrete, as was done on the slopes, would doubtless prove much more effective under trying conditions. The author is much interested to learn through Mr. Hering's comments that the slopes of the Queen Lane Reservoir at Philadelphia are to be treated in a manner similar to the Astoria work, especially as he knows that the engineers charged with the former work have given much time to the investigation of different types of asphalt linings, and have doubtless had to do much sifting in the determination of the actual facts relating to the matter.

Regarding the proportions used in making the concrete, the writer is not disposed to agree with Mr. Gould in pronouncing the composition a lean mixture. The apparent scarcity of sand has been accounted for by the presence of a large amount in the gravel used. The size of this gravel varied from that of a pea to fine sand. The volume of the voids in the sand and gravel was carefully determined, and the requisite amount of cement thus ascertained. In reality, the proportion of sand to cement was little in excess of two to one. The resulting large saving in cement was accomplished by the use of gravel in addition to the broken stone, an economy overlooked by many engineers; and the proportion of cement to voids was doubtless greater than in the three-to-one mixture which Mr. Gould speaks of having used so successfully at Havana, Cuba. The resulting concrete was excellent in quality, and could be picked with the greatest difficulty after being in place ten days. The honesty of the contractor in the use of the cement was no doubt due to its being furnished by the city.

Mr. Gould's statement that the stave pipe shows but 5% advantage over the riveted pipe, because this amount of increase in the diameter

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of a riveted pipe would give a carrying capacity equal to the stave pipe, seems rather unfair to the latter, since any increase in diameter renders necessary a proportional increase in the thickness of the metal also, if the same factor of safety is to be preserved. The ratios of their respective carrying capacities for similar diameters, as determined by use of the experimental value deduced for the factor n for each class of pipe, would express their real relation much more correctly, and in this case would show an advantage of 12% in favor of the stave pipe. Mr. Adams.

Mr. Henny calls attention to the probable economy resulting from the use of a broken hydraulic grade line instead of a uniform diameter of pipe. With steel pipe there may often be effected a considerable saving in this way when dealing with light pressures and pipes of moderate diameters, in which cases the requisite thickness of metal is determined by structural requirements rather than the pressure. Under these conditions an increase of diameter and an incidental saving in fall can be accomplished without increase in the thickness of the plate; and the head thus saved may be used, as Mr. Henny suggests, in decreasing the diameter where the pressure is the controlling factor in determining the thickness of the metal. Though true to a less degree than in the case of steel, wrought-iron or cast-iron pipes, this reasoning is also applicable to stave pipe and applies at Astoria.

The development of power from the pipe line was wholly incidental to the general design, which was controlled by other considerations, principally the desire to supply directly by gravity three of the four distributing services contemplated. The only additional expense incurred by the power appendage was due to the fittings and building necessary for its utilization.

There seems to be some difference of opinion between Mr. Riffle and Mr. Henny as to the value of published statements of cost of work in general, but there seems to be no disagreement regarding their value to those who know how to use them. It is therefore fortunate in the present instance that the tables of cost have been submitted to the American Society of Civil Engineers.

Mr. Riffle and Mr. Harlow both take strong exception to the author's defence of awarding contracts to the lowest bidder. Mr. Riffle's argument would in the case in hand be a very plausible one, were his premise, that "the contractor had no rights, either moral or legal, to enforce," a correct one. The very bad previous reputation which he gives the contracting company in question, based, he says, on personal knowledge, was, unfortunately for his argument, not borne out in the careful investigation made by the Astoria Water Commission previous to making the awards. In justice to the contractor in question, it may be said that these damaging facts proved to have very slight foundation when divested of the superstructure imposed by competing contractors. As a guarantee of the company's ability to furnish satisfactory sureties on its bonds, it had deposited with its

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Mr. Adams. bid a certified check for $11 000, although the total amount of the contract awarded it aggregated but $50 000. Whatever the final outcome, it seems manifestly unfair to designate such a company as an irresponsible bidder; and the author, after carefully weighing all facts, was in accord with the Commission in determining that the company in question did not clearly belong in that category, and, as the lowest bidder, had rights that should be recognized. Had this bid been rejected under existing circumstances on the ground of irresponsibility of its signers, and the work awarded at a higher figure to other parties, the public as well as the disappointed contractor could very properly have demanded sound reasons for this repudiation of all that is implied in the advertising of work to be let to the lowest responsible bidder, and the requiring of certified checks, bonds, etc., as a guarantee of that responsibility. If, as Mr. Harlow favors, contractors are not to be awarded work because, regardless of security asked and offered, the engineer considers the bid too low, then the whole system of competitive bidding and of requiring guarantee bonds from the party to whom award is made, is but foolishness, since the requiring of security means nothing. Consistency will require those not satisfied with present methods of public contract letting to advocate radical fundamental changes, rather than to wish the logical sequences of those methods ignored, or to expect engineers in charge of work to make themselves unwelcome and self-constituted guardians of the private fortunes of contractors and their sureties who are so bold as to differ from them in their ideas relative to the probable cost of work. The author believes that the interests of contractors of unquestioned responsibility are best protected against men of doubtful standing by just such an unswerving, consistent course as that adopted by the Astoria Water Commission, by which means, without in any way jeopardizing the city's interests, a financially irresponsible contractor, should he prove to be such, would be speedily and permanently disposed of as a future competitor. At Astoria, thanks to the exercise of care in the acceptance of bondsmen, there were no expensive delays incident to the contractor's failure, and this part of the work was completed sooner, and on the whole with less trouble, than was the case with certain financially very responsible contractors on other parts of the work. The $4 500 "additional cost to the city" above that earned by the contractor on the reservoir was not, as Mr. Harlow suspects, used in settling unpaid bills of the contractor, but was the price paid for the cement used in the work.

Owing to the geological formation at Astoria, the unstable ground all lies upon the north or Columbia River side of the peninsula, while on the south or Youngs Bay side the material is of a wholly stable character. It was because of this condition that the reservoir was located on the south side, and the connection accordingly made with the distribution by means of the tunnel.

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TRANSACTIONS.

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No. 784.

THE CONSTRUCTION OF A LIGHT MOUNTAIN RAILROAD IN THE REPUBLIC OF COLOMBIA.

By E. J. CHIBAS, Assoc. M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, 1896.

WITH DISCUSSION.

In the construction of a railroad in a country where sufficient traffic is already in existence to warrant a fair return on the contemplated investment and where capital can be easily procured, the main object should be to build the road with such light curves and grades that the operating expenses may be kept down to the lowest limit, almost regardless of first cost, as long as the interest on the additional capital required does not exceed the saving that will be effected in the operating expenses by the additional outlay, as in that manner the line will be in a condition to face active competition. On the other hand, where a railroad has to be built in a new and undeveloped country to accommodate a light traffic which is not expected to increase materially for many years to come, and where capital is limited, it becomes necessary to adopt sharper curves and heavier gradients than customary, if, by so doing, the first cost of construction can be decreased to such an amount as to make possible an enterprise of otherwise

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prohibitory cost. The purpose of this paper is to describe a railroad of this nature, built for the Caribbean Manganese Company of Baltimore, Md., in the Department of Panama, Republic of Colombia. The main line of the road at present in operation is 9 miles long and extends from the harbor of Nombre de Dios to the Nispero and Soledad manganese mines. The Nombre de Dios Harbor is on the Caribbean Sea, about 40 miles northeast of Colon.

During the spring of 1892 the author was requested to make preliminary surveys and estimates for a railroad from the mines which had recently been discovered, to the nearest and most appropriate place on the coast available for shipping purposes. An examination of the coast showed that there were only two places to be considered, Viento Frio and Nombre de Dios, the former being 5 miles east of the latter. Both were inhabited by a few native villagers whose only communication with the outside world was by means of their canoes, as there was no vessel running to either locality, except an occasional trading schooner which would stop far from shore and take in a small load of cocoanuts or ivory nuts. No plans or chart of that portion of the coast having ever been made, approximate hydrographic surveys of both harbors were made to determine their future possibilities as shipping ports. The investigation revealed that Nombre de Dios would make a fairly good port, while Viento Frio could not be used at all unless a large amount of coral rock was blasted and dredged.

A general exploration or reconnaissance of the country lying between the mines and the two ports was then made, which was a very difficult undertaking, as the country was covered with a dense forest or thick undergrowth, the larger portion of which had never been penetrated, except, perhaps, by a few natives. There were no roads and very few footpaths. The investigations had to be carried on generally by wading and following up the streams and creeks, by ascending to the top of adjacent hills or climbing trees at convenient localities from which to ascertain the topography of the surrounding country. It was finally decided that the most economical route for the railroad would be the one following the general direction of the Viento Frio River, as it was discovered that between the mines and the sea there are ranges of hills running parallel to the coast which only appeared to flatten out when they approached the narrow valley formed by the Viento Frio River.

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Preliminary lines were then run from Nombre de Dios and Viento Frio to Nispero along the general route selected by the reconnaissance. The progress of this survey was very slow, as it depended almost entirely on the progress made by the natives in cutting out the line with their machetes through the thick undergrowth. The average progress varied from 2 000 to 4 000 ft. per day. Topography was taken on both sides of the line, covering a belt of country from 200 to 500 ft. in width. The whole line was afterward plotted on a scale of 400 ft. to the inch. The line from Viento Frio to Nispero turned out to be 6¼ miles long, and the one from Nombre de Dios 8¼ miles. The former was the shorter and cheaper line, but the saving in its cost was far from compensating for the large expenditure which would have to be made to improve the Viento Frio harbor. For that reason the author, in his report made to the company in July, 1892, advised against the construction of that line, and, after weighing all considerations, recommended the construction of the railroad from Nombre de Dios as the more economical line.

During the latter part of December of the same year he was asked to take charge of the work to be carried out in Colombia, and was informed at the same time that there had been many difficulties in obtaining the right-of-way to Nombre de Dios, so the company was forced to build the railroad to Viento Frio. As no improvements in the harbor were contemplated, the ships would have to anchor outside and receive their cargoes by lighters. During some portions of the year this was possible, as the dead calm prevailing in that latitude made the sea very smooth, but at other times the prevailing trade winds made the sea so rough that it would have been impossible to ship at all. For this reason it was decided to send only a few shipments from Viento Frio, in the hope that the right-of-way matter would be settled in the meanwhile, and then proceed to Nombre de Dios.

During the latter portion of 1892 Mr. E. G. Williams had been sent by the company to make surveys from Viento Frio to Nispero along more direct routes; but his estimates proved that any such routes would cost twice as much as the one proposed by the author.

The location of the railroad was therefore made along the Viento Frio River route, and it was started during the first week of February, 1893. The average rate was in the neighborhood of a quarter of a mile per day. Although the plans of the original preliminary

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were carried in the field for constant consultation, no attempt was made to follow the paper location. Maximum curves of 40°, 146 ft. radius, and maximum grades of 5% in favor of traffic, were adopted, so as to reduce to a minimum the amount of excavation to be made. The only adverse grades were some short ones of 1 and 1½ per cent. As it was only in the last mile that the natural level of the country rose very rapidly, it was possible to concentrate all the heavy grades in that portion of the line, which made operating very convenient, as by placing a switch at the foot of the grade the locomotive can haul to it a large number of empty cars and from there to the mines make as many trips as the number of sections into which it is found necessary to divide the train.

The country being broken by a large number of gulches and creeks, a corresponding number of small bridges was required to cross them. All the drainage of the portion of the country under consideration is carried to the sea through the Viento Frio River. This has a very moderate velocity in ordinary stages and is only from 40 to 80 ft. in width, but during the rainy season and in time of flood, like most of the tropical streams, carries a large volume of water with a corresponding increase in velocity. The located line crossed this river twice, and its tributary, the Mamey, four times. In all these cases it was found cheaper to bridge across than to keep on the same side, as either the increase in the length of the line resulting from following the windings of the stream or the increase in excavation due to the much rougher mountain side would have added considerably to the expense of construction. In locating the line along the side hills it was fitted as close as possible to the natural formation of the ground, and it was placed in cuttings so that the whole road-bed would rest on solid ground, as no embankment could have stood on the steep side hills, washed at the bottom by a stream that would carry away any embankment, not formed of broken rock, which would come within the limits of its high-water mark.

After the location was completed the older estimate was revised and a new and careful estimate was made of the cost of constructing the railroad from Viento Frio to Nispero, including the equipment necessary to begin operations at the mines, as well as the lighters required at the port for shipping the ore. This estimate had to be approved by the Board of Directors of the company in Baltimore before any

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work of construction could be started, but while waiting for further orders, the clearing and grubbing of the line was executed and work suspended in May. Construction was finally started in January, 1894. It was found that the right-of-way which had been cleared and grubbed seven months previous was covered with luxuriant vegetation, attaining in places heights of 5 to 7 ft. Some of the center stakes furnished proof of the extreme vitality of the vegetation in that portion of the tropics, as they seemed to have been imbued with new life and were covered with green leaves.

After the clearing had been performed several places were found in going over the line, as is usually the case, where improvements could be made in the alignment, and changes were made accordingly, although not as many as desirable, because the work of grading was pushed so rapidly as to give the engineering corps no time for making minor changes. The last mile, however, comprising the most difficult portion of the work, was gone over. The previous work was platted to a scale of 100 ft. to the inch. A new location was made on paper, and trial lines following the paper location closely though not exactly were afterward run on the ground until the result was as satisfactory as the nature of the country would allow.

It was believed that the cargo of material, comprising rails and fittings for 6¼ miles of track, timber, cars, a locomotive and other items, would arrive about May 1st at Viento Frio, but various causes delayed the accumulation of sufficient rails and timber to begin work until about August 1st, when track-laying and bridge-building were started.

The bridgework was a source of considerable trouble, as the chief carpenter expected from the United States never arrived. None of the so-called carpenters engaged in the neighborhood could understand or work from plans, and their workmanship was so poor that, even after laying out the whole work for them, they had to be watched closely to keep them from spoiling the timber. It was not until after half of the work had been accomplished in this manner that a few fairly good carpenters were secured; the work then proceeded at a better rate and with little interruption.

Owing to these difficulties, it soon became evident that the first shipment of ore could not be sent from Viento Frio about the middle of September, as had been expected, and if the shipping was delayed

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rainy season had settled in, that fevers began to spread, and a month later there was not an employee, native or foreign, who had escaped them. The Jamaica negroes suffered less from their effects than any of the other laborers. These fevers were of a malarial nature, and returned at intervals of four to five weeks, according to the individual. They did not assume a malignant nature and no fatalities resulted from them, but they were a source of considerable annoyance and directly or indirectly caused frequent interruptions in the work. Whenever work of this nature is performed in the tropics more or less fever is apt to develop; it cannot be avoided, but with good care and by the enforcement of sanitary regulations, it can be prevented from developing into a dangerous epidemic. This trying period only lasts during construction.

Handling Ore.—The main manganese ore deposit lies in the district enclosed by creeks *A* and *E*, Fig. 1. The former is in what is called the Nispero mine, and the latter in the Soledad. The distance between both creeks is about half a mile. From both the Nispero and Soledad terminals of the railroad the ground rises very rapidly toward the summit marked *S* in the plan. This point is about 2 000 ft. horizontally from the Nispero terminal of the railroad and 500 ft. above it. The flanks of the mountain are cut up by several narrow gulches and ravines. For the transportation of the ore to the railroad cars, iron side-dumping mining cars of about ½ cu. yd. capacity are used, running on light portable track. The first track laid started from the top of the ore pocket at the Nispero terminal of the railroad. It was built along the side hill with a light rising grade of from 1½ to 2%, so as to allow the loaded cars to go down almost by themselves and at the same time allow the empty cars to be pushed up without much exertion. Similar lines were graded and laid at vertical intervals of 25, 50 or more feet, as the distribution of the ore required them. Connecting all these lines there is an inclined gravity plane with a grade of 30%, and as the loaded car goes down, it pulls up the empty one. The mining cars are loaded at each different level and pushed to the inclined plane, where they are hooked to the end of the rope and let down to the lower level, when they are unhooked and pushed to the ore pocket and their contents dumped.

In several cases where bodies of ore prevented direct communication for some time between the side lines and the inclined plane, the

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PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXVI, No. 784.
CHIBAS ON CONSTRUCTION OF MOUNTAIN RAILROAD.

FIG. 1.

FIG. 2.

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cars, instead of going down the incline, dumped their contents into a chute with an inclination of 45° to allow the ore to roll down to the bottom, where by lowering the gate its contents dropped into the car standing at that lower level or into empty cars standing on the incline. This system has not been extended to more than 150 ft. above the Nispero railroad terminal, as the mining operations are confined to that belt, but the same general system could be extended to cover the whole deposit and by close attention to details the number of handlings of the ore could be reduced to a minimum. After the work becomes higher more economy in operation would probably result from using two self-dumping cars on the gravity incline and confining each car to its own level. The author believes that the use of a wire cable tramway in this case, beside involving a larger expenditure in first cost, would not offer any economy in operation, considering the wide belt over which the ore is spread.

The ore pockets at the Nispero terminal of the railroad hold about 75 tons of ore, and those at the Soledad terminal about 200 tons. The inclination given to the bottom of the ore pockets, floored with 3-in. planks, was 45°, as this was found to allow the ore to slide down without sticking to the plank. Both ore pockets are provided with enough chutes to allow four or five railroad cars to be loaded at once. Plate III, Fig, 2, shows a portion of a train carrying 50 tons of manganese ore, coming down the 5% grade.

The natural conditions of Nombre de Dios Harbor were very favorable for building a wharf at a small expenditure, as the deep water coming very close to the shore, which is of coral formation, allows the construction of a wharf with its length parallel to the shore and from only 30 to 50 ft. wide. Its length is about 200 ft. The steamers can load at the wharf as much as they can carry on a 17-ft. draft, which generally varies from 1 300 to 1 800 tons. To send a larger cargo the remaining portion is lightered to the steamer, which must anchor about 1 500 ft. from the wharf. At the port terminal of the railroad the ore is dumped off the cars from a trestle 10 ft. high running nearly parallel with the wharf and about 100 ft. from it. The manganese is loaded in ore buckets carried on small car trucks which are pushed on rails to the side of the steamer, where she lifts them with her own winches. In this manner by using sixteen buckets, it was possible to handle 400 tons per day of 10 hours, by loading through

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two hatches at the same time. By a few contemplated improvements the rate of loading per day could be increased.

The first locomotive in use weighed 25 000 lbs., distributed over two pairs of driving wheels measuring 30 ins. in diameter. The cylinders were 10 ins. in diameter by 14-in. stroke. The wheel base was 5 ft. 3 ins. No difficulty was encountered in running around the sharp curves with this locomotive, and after a year of constant use it did not show any effects of hard usage. Many, but not all, the 40° curves are provided with inner guard rails.

Water is pumped to a water tank in the lower portion of the line by means of a Rider hot-air engine, which proved very serviceable, as there is no boiler to look after and it needs very little care and skill to run it. Both of these considerations are very important factors in those countries where skilled labor is difficult to find and comes very high. The pump in question has a capacity of 400 galls. an hour and has been in use for over a year and a half without giving any trouble.

Cost.—In giving the cost of the various items of the work already mentioned, no portion of the engineering or other general expenses were charged to them.

The average cost of the railroad constructed from Viento Frio to Nispero was $12 000 per mile, the average for the Nombre de Dios branch was $6 800 per mile, including the cost of the wharf, of buildings and dwellings erected at Nombre de Dios, and the improvements at the port terminal. The cost of the 10½ miles of railroad built and equipped averaged $13 400 per mile. This included, beside the equipment of the railroad proper, thirty mining cars and ½ mile of portable track for the mines, as well as some harbor equipment, such as four 30-ton coppered wooden lighters, two iron buoys, two hoisting engines and derricks for the wharf. The cost of the wharf, buildings, etc., is also included, as well as that of the ore bins, and in fact all expenses incurred, whether in South America or Baltimore, in placing the entire property in working order from the time the author was authorized to begin construction in January, 1894, to December, 1895.

The heaviest work, as well as the most expensive portion of the railroad, was the last half mile, which cost $9 000. The amount of material handled in the grading of this section was 22 000 cu. yds.

The actual cost of the railroad from Viento Frio to Nispero, including equipment for railroad, mines and harbor, turned out to be only

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1¼% greater than the estimates made by Mr. E. G. Williams and the author in the spring of 1893, to which reference has already been made. It is fair to state, however, that the unit prices did not agree. Some of the items, like grading, ties and bridge buildings, came out lower than had been estimated; but on the other hand, the general expenses and contingency accounts were higher. The excesses and deficiencies, however, more or less compensated each other, with the final result stated. Changes were also made in the work and equipment, but they compensated each other and had no effect on the final result.

In making estimates for work to be performed in the tropics it must be borne in mind that in starting enterprises in these countries many expenses have to be incurred that would be foreign to a similar enterprise in the United States. It is true that labor is generally cheaper per day, but not so in view of the amount of work it accomplishes, with few exceptions in some particular lines which do not count for much in the long run. A doctor must be kept on the ground. The hospital account must be considered as well as its indirect effect in contributing to lengthen out the work and decreasing the value of physical exertion. Due allowances must be made for the transportation of the material to the nearest port and also for local transportation, which is generally very poor. There are also many other minor considerations inherent to all new enterprises in a new country, which, if not fully provided for, will overrun the estimated contingency account.

The author is pleased to acknowledge the valuable co-operation of the engineers associated with him in this work. He has already mentioned the connection of Mr. E. G. Williams with the early stages of the work, with whose able co-operation the first location of the railroad from Viento Frio was made. For all the work performed and described in this paper from the time construction was inaugurated, the writer is indebted to Messrs. R. M. Arango and C. C. Arosemena. Both were connected with the company from January, 1894, the former having been Principal Assistant Engineer to January, 1895, and the latter from that date to July of the same year.

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CORRESPONDENCE.

Mr. Cummings ROBERT A. CUMMINGS, Assoc. M. Am. Soc. C. E.—In recent work with unskilled labor in a semi-tropical region, under similar conditions to those described in the paper, the method of paying the men bi-weekly at a fixed rate per day was too much of an innovation to satisfy the natives. Daily and monthly payments were tried with equal inconvenience to employer and employee. Finally task work was adopted and considered satisfactory by the men, although not entirely so by their employer, as little economy resulted. Then a system of rewards for rapid and efficient task work was introduced, the rewards being inexpensive things regarded as luxuries by the men, and in this way a saving of 10% to 50% was effected, the gain depending in part on the individual and in part on the nature of the award.

In making preliminary surveys through such heavily timbered regions as described by the author, the writer has recently obtained very satisfactory results by using a surveyor's compass and a 100-ft. chain for running angle and grade lines, and by using a clinometer for cross-section work. On these surveys, which extended several miles through dense woods and swamps, the writer acted as his own instrumentman, with natives as chainmen. The work checked up quite accurately for the purposes of the survey.

With regard to the slides that are stated in the paper to have occurred in the cuts after construction was completed, the writer has found that the heavy clays from which a seepage of water took place were invariably troublesome, and required a slope in some cases of four to one, depending on the degree of saturation. When the clay was stiff and contained a normal amount of moisture, a nearly vertical cut was preferable, being less exposed to the elements. The dry cuts usually left by steam shovels maintain themselves and are frequently found unchanged after the lapse of a considerable period, which is thought to be due largely to the verticality of the slopes.

As supplementary to the author's notes concerning timber, it may be stated that the *torregia taxifolia*, or stinking cedar, found along the Appalachicola River in Florida, has a wood remarkably durable when exposed to the warm, humid climate of the district. The life of fat yellow pine is but six or seven years, whereas this cedar is unappreciably affected in the same length of time. A bottom rail in a worm fence dividing a swamp from hummock land, which was removed after being in place 38 years, was still in good condition.

Mr. Gould. E. SHERMAN GOULD, M. Am. Soc. C. E.—As the author truly remarks, the aim in the construction of such roads as he describes is to have the line established, and trains running over it, as quickly and cheaply as possible, consistently with reasonably permanent installa-

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tions. In the case of the road described in the paper, the amount of traffic could be estimated in advance more easily and accurately than in the general case of a road built to develop a business not already existing. With the latter class of lines, cheap and rapid construction is highly desirable in order that the paying possibilities of the roads may be demonstrated. It is generally difficult to raise the money for a projected road the profits of which exist only in the prospectus of the promoters. If even a temporary line can be laid and operated with large earnings, there will be far less difficulty in placing bonds for improvements and extensions than in attempting to bond a mere concession for a road not yet built. Mr. Gould.

It is to be regretted that sufficient time has not elapsed to determine the relative durability of the native timber and Georgia pine. One serious difficulty in making preliminary estimates on such work as that described in the paper is due to the meager information concerning the cost of timber, particularly ties. It is hard to arrive at a reasonably accurate estimate of the cost of the native timber, even from information obtained on the ground, because when the time comes for making contracts prices may be expected to vary considerably from those given first. The author states that ties of native timber averaged 25 cents apiece, which is remarkably low, but it appears many decayed very rapidly, so it is fair to conclude they were of inferior wood. On the other hand he says lignum vitæ ties on the Panama Railroad cost as high as $1 80 per tie, which price, in connection with their great weight and the cost of boring holes in them for spikes, makes their use very expensive. If it is found that ties of Southern pine, either in an untreated state or creosoted, will resist the attack of tropical ants for even two or three years, this knowledge will greatly facilitate the making of correct estimates for such roads.

The author indicates correctly that estimating on work to be done in the tropics is quite different from figuring the cost of similar work in the United States. It requires considerable experience, the introduction of a liberal estimate for contingencies, or both, to provide for the unforeseen conditions sure to be found, and it is best to make allowances for these from the outset.

José R. Villalon, Assoc. M. Am. Soc. C. E.—It is generally considered by people living in temperate climates that the moon does not exert any influence on timber, and that the belief in this influence, so common in tropical countries, is only a superstition held by uneducated classes. The following remarks show that the belief is not a superstition, but knowledge gained from actual observation: Mr. Villalon.

A few years ago while the writer was engaged on the Antioquia Railroad, in the United States of Colombia, it was necessary to pass through country where water was scarce and unfit for drinking. Under these circumstances it was customary to search for a large

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Mr. Villalon. species of bamboo about 8 to 10 ins. in diameter and 2 ft. from joint to joint. If the moon was in the first half, plenty of water was always to be had by making a hole just above a joint, so as to empty the space within the cane between that joint and the next above. When no water was to be had from the bamboo in this locality, the moon was invariably in its last quarter. To secure drinking water in other localities along the same railroad, a species of vine about 3 or 4 ins. in diameter was cut into pieces about 3 ft. long; if this was done during the first half of the moon one or two glasses of sweet pinkish sap was obtained, although the yield was very slight during the last quarter of the moon. The writer had a similar experience with another vine in Cuba.

It is a well-known fact in Cuba and Colombia that bamboo, vines and various relatively soft woods, when cut during the first half of the moon do not last over six months on account of the rapid fermentation of the sap, while if cut during the last quarter they remain for years in very good condition. The durability of timber cut during the waning moon depends greatly on its age; it lasts longest when neither too young nor too old. The same effect is clearly seen in the case of structural timbers; the soft part containing the most sap is the first to decay, if cut during the first half of the moon, while the heart is attacked more slowly. Timber felled during the last quarter lasts a very long time. Not only is the wood subject to this influence of the moon, but leaves, stems and other parts of the tree.

The palm is one of the most useful trees in the tropics; many portions are employed in the construction of country houses, the outer part is used for boards, its leaves serve for thatch, and the leaf stalks are used to cover the outside of houses, for packing tobacco in bales, and for many domestic purposes. It is always the case that if the tree is felled or the leaves cut during the first half of the moon, only a few months of serviceability are to be had, while if cut during the last quarter a service of ten or twelve years may be expected. The writer is reliably informed that in one instance a thatch made of palm leaves cut during the last quarter of the moon was in good condition 32 years after it was placed.

This practice of timber felling is so general in the tropics, and is so decidedly approved by all classes, that in all contracts and specifications a clause is generally inserted requiring the timber to be felled during the waning of the moon; when not so stated, it is always understood. In some cases the timber is felled one or two years in advance, not only to have it well seasoned before use, but also to detect, by its decay, if it has been felled during the first half of the moon. Mr. J. M. Fernandez, in his book on Cuban agriculture, states that the felling of trees during the last half of the moon is clearly required by experience in such matters on the island, and

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recommends doing this work during the last quarter. It should also be remarked that in an order to the French Forestry Commissioners, Napoleon I directed the felling of trees for naval timber to be done only between November 1st and March 15th, and during the waning moon. Mr. Villalon.

These facts warrant the following conclusions:

First.—In the tropics the moon exerts an influence on the circulation of the sap in trees, there being the least circulation while the moon is waning.

Second.—In the tropics, where the difference between the seasons is hardly noticeable, the circulation of the sap is continuous, the rate varying with the quarter of the moon.

Third.—It being the universal practice to fell trees for timber when the wood contains the least sap, it follows that the tropical practice of doing this work during the waning moon is fully justified, and agrees with the practice in more temperate climates of doing the same work during the winter.

E. J. CHIBAS, Assoc. M. Am. Soc. C. E.—The remarks made by Mr. Cummings concerning slides and the stability of slopes in clay cuts agree with the observations of the author. Two of the slides on the road described in the paper were not due to the character of the clay itself, however, but to the presence of a thin, inclined layer of permeable material embedded in it. After heavy rains, water passed through this layer and seemed to give it somewhat the character of a lubricating material, causing the upper bed of clay to slide over the lower practically motionless mass. The layer of permeable material was only a few inches thick, and could be traced easily along the sides of the cut. Mr. Chibas.

Mr. Villalon's observations concerning the influence of the different phases of the moon on the circulation of sap in tropical plants substantiates the author's statement in the footnote on page 77. It might be added that persons engaged in the rubber industry in Colombia always tap the rubber trees during the period preceding the full moon. They claim repeated experience has taught them that the sap is then more abundant, and that they obtain more than if the operations were carried on during the waning moon. All agree that to obtain the best timber trees should be felled when the vegetation is at rest. In temperate countries this happens during midsummer and midwinter, and, therefore, these two periods are selected for such work. In the tropics, where there is no appreciable difference in temperature throughout the year, the seasons seem to have little influence on the circulation of the sap, the minimum quantity being found about once a month. This condition of minimum circulation appears to coincide always with the waning moon, so it is fair to conclude that the moon exerts a certain influence on the circulation of the

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Mr. Chibas.

sap. Since the paper was written the author has been informed that the practice of felling trees for timber during the waning moon is not confined to the tropics, but is also followed to some extent in parts of Southern Europe, particularly in Italy.

With reference to Mr. Gould's remarks, the author would state that ties of Southern pine will last for two or three years in the Republic of Colombia, and at least double that time if they are creosoted. As none of the available timber around Nombre de Dios had been employed for ties, it was necessary to experiment with several kinds. Some proved to be of a poor quality and decayed within a year, but from all appearances a large majority will probably outlast the more costly Southern pine.

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No. 785.

THE CONDITION OF STEEL IN BRIDGE PINS.

By A. C. CUNNINGHAM, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, 1896.

WITH DISCUSSION.

The engineer who receives a report of a large number of tests from plates, angles and other shapes used in his bridge, which indicates an excellent material, may naturally suppose that his bridge pins, made under the same specification, and perhaps from the same casts of steel, are of the same excellent quality. That such is not the case will be demonstrated.

A few years ago it was customary to accept bridge pins, and also other material, on the result of a test made on a specially rolled ¾-in. round rod. The apparent simplicity and convenience of this method of testing was what led to its adoption, but it did not take long to demonstrate its inconvenience, and the fact that the steel was often in finished shapes before the test rods were rolled. This resulted in the substitution of tests from the finished material, with or without the round tests as a matter of record, and this substitution soon dem-

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onstrated that a small round test was not an accurate indication of the condition of steel in other shapes.

Very naturally this replacement of round tests by tests from finished material led engineers to ask for tests from finished pins, but such requests did not meet with encouragement from manufacturers, who opposed them on the grounds of expense and time. The engineer finally prevailed, however, and pins began to be tested, with results startling to some and disagreeable to others. The test from finished pins is now an established, though not a popular, custom.

Bridge pins are of two general classes, rolled and forged, and the change from one class to the other takes place at a diameter of about 6 ins. An ingot is rolled down to a bloom of convenient size, and this bloom is rolled or forged to the size ordered.

In the case of rolled pins the main consideration of the roller is to finish the pin truly round and of the required diameter and not break his rolls. This last consideration leads him to have the bloom heated as hot as may be without burning, and in consequence it happens frequently that some blooms are too hot and are burnt or overheated. Six-inch pins are often finished so hot that a bar 25 ft. long can be cut into 18-in. lengths on the hot saw. It will be seen from this that heat treatment and finishing temperature are refinements which stop short of rolled pins. These are generally made from the smallest blooms that are suited to the rolls and will finish to given diameters, and are finished very hot. In consequence, they lack work, both in reduction of section and finishing temperature, and are likely to be coarsely crystalline and deficient in toughness and ductility, if not absolutely brittle; see Tables Nos. 2 and 3.

The main reason for changing from rolled to forged pins at a diameter of about 6 ins. is a practical one. Existing rolls will not make a much larger size, and in the lengths in which they are rolled, they become difficult to handle. In making forged pins, the size of bloom is again the important consideration for the manufacturer. It must not be too large for the hammerman to handle conveniently in the heating furnace or under the hammer, and the result is that many so-called forged pins are little more than blooms with the corners knocked down. The weight, power and size of the hammer used for making forged pins have an important bearing on their final condition. For a small hammer a small bloom must be used, and it must

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be made very hot. The blow from the small hammer on a large bloom is merely a surface blow, the work of which does not penetrate the metal to any extent.

With small hammers and large pins there is, then, a highly heated bloom, the shape of which is changed to a round by sliding the exterior over the interior without working the latter, and with the result of frequently rupturing the interior portions, especially if the bloom is not uniformly heated. The larger the hammer used for making forged pins, the better will be the results; larger blooms can be used, the work of the blow will penetrate deeper, they can be forged at a lower heat, and some attention can be given to the finishing temperature.

When pins first began to be tested, it was quickly discovered that the test nearest to the surface of the pin gave the best results, and that there was a constant falling off of quality as the test approached the center. It was also discovered that the size and shape of the test piece affected the results. Of two test pieces cut near the surface of a pin, one ½ in. in diameter, and the other 1 in., the former will give the better results, and a flat, thin test piece in the line of a chord will give better results than either of the round pieces.

The most marked difference between the tests from pins and tests from shapes or plates is in the reduction of area and the appearance of the fracture. While the test piece from the surface of a pin may give a fair reduction of area with a fine, silky fracture, as the center is approached the reduction becomes decidedly less, and the fracture grows more coarse, irregular and cokey, until it may finally terminate with a crystalline fracture and little or no reduction. This change of appearance and loss of reduction indicates lack of work, too much heat, or both.

When pins are tested strictly with a view to ascertaining their quality, and not for the purpose of getting them past a specification, it is found that forged pins are better than rolled pins, and that the better the steel, the better the resulting pin, other conditions being the same. It is also found that a medium hard steel of about 70 000 lbs. ultimate strength will give better results than a soft steel. All pins are also improved by annealing.

From a consideration of Table No. 1, Cast A, it will be seen that even with an excellent quality of uniform steel, there is a falling off

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of ultimate strength and physical properties as the center is approached. This may be ascribed to a lack of work and a high finishing temperature. Table No. 1, Cast B, shows the possibility of having highly segregated steel in a pin. In this pin the phosphorus and sulphur have doubled from the outside to the center, and from an excellent quality of steel the metal changes to a very ordinary quality. Had the cast analysis of this steel shown only an ordinary quality to start with, the steel at the center of this pin would have been dangerously defective.

Table No. 2 shows the generally poor results in a rolled pin made from common steel, and how the test may be improved by annealing. It may fairly be assumed that the pins themselves would also be improved by annealing. In this table, as in some of the others, there is what appears to be a paradox, namely, a rise in ultimate strength after annealing. This is undoubtedly due to a release of initial stress in the original pin by the annealing. A rise of ultimate strength, and an improvement of elongation, reduction and appearance of fracture after annealing are not confined to pin tests; the same phenomena are observed in eye-bars and other heavy sections. The initial stress may be readily caused by uneven heating or cooling.

Table No. 3 shows how un-uniform and treacherous it is possible for rolled pins to be. This steel is of a common quality, and not by any means as poor as much that is made. The analyses from the broken end indicate that it was approaching the zone of maximum segregation, but stopped short of it. From this table the improvement due to annealing may be again seen.

Table No. 4 shows the improvement in physical properties due to annealing the test piece, and, also, to even a crude annealing of the pin itself. It also shows that a good analysis does not alone insure good results. The blooms from which these pins were forged were so small that they received but little work, and were finished at a high heat.

From the foregoing it may be seen that bridge pins, even when made from a superior quality of steel, are not in a uniform condition; that the interior portions show a lack of work, and may show excessive segregation, which in steel of common quality is liable to reach a dangerous amount; it is also shown that pins are improved by annealing.

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The following specification is submitted as being likely to procure as good bridge pins as can be had in common practice:

Bridge pins shall be made of open-hearth steel.

If made of acid steel, the cast analysis shall show not more than 0.06% of phosphorus, nor more than 0.08% of sulphur. If made of basic steel the cast analysis shall show not more than 0.03% of phosphorus, nor more than 0.03% of sulphur.

The manganese for either steel shall be not less than 0.5%, nor more than 0.8 per cent.

The smallest diameter of the final bloom or ingot to be made into pins shall be at least 50% greater than the diameter of the pin.

All pins shall be stamped with the cast number of the steel from which they are made, and shall be stamped with consecutive numbers from the bottom of the ingot to the top.

After pins have been manufactured to diameter, they shall be slowly and uniformly heated to a dark red heat in a suitable furnace, and left to cool slowly for a period of not less than 24 hours.

Tests cut from near the surface of annealed pins, and of a sectional area of not less than ½ sq. in. shall have an ultimate strength of not less than 60 000, nor more than 70 000 lbs., per square inch, an elastic limit of not less than one-half the ultimate strength, an elongation in 8 ins. of not less than 20%, and a reduction of area at fracture of not less than 40 per cent.

Tests cut from near the center of pins shall comply with the above requirements, except that a fall of 5 000 lbs. in ultimate strength, and a fall of 5% in reduction of area, will be allowed.

The fractures of all pin tests shall be free from granulation.

Analysis of drillings taken from near the surface of pins shall not exceed the limits of the cast analysis by more than 10%, and, when taken from near the center, shall not exceed said limits by more than 50 per cent.

Tests and analyses shall be made on such pins as the engineer shall designate.

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TABLE No. 1.—Tests from Acid Open-Hearth Steel Pins.

Nomenclature for Fractures.—C., Cup; Ck., Cokey; Cr., Crystalline; I., Irregular; S., Silky.

Cast.	Chemical Test. Percentage of elements. C.	P.	Mn.	S.	Test From	Physical Test. Stress, pounds per square inch. Elastic.	Ultimate.	Percentage of strain. Elongation in 8 ins.	Reduction in area.	Nature of fracture.
A	.22	.051	.39	.031	Cast analysis.					
"	.22	.049	.39	.030	10-in. forged pin No. 1.	37 780	62 550	20.7	50.0	Ck. ½C.
"	.22	.050	.40	.031	" " " No. 2.	37 800	58 780	19.2	43.0	" "
"	.28	.053	.39	.035	" " " No. 3.	36 570	56 140	22.5	37.7	" I.
B	.22	.053	.40	.038	Cast analysis.					
"	.24	.053	.44	.033	8-in. forged pin No. 1.	39 820	60 640	20.2	52.9	S. I.
"	.28	.062	.45	.039	" " " No 2.	41 700	66 200	18.7	38.7	Ck. I.
"	.38	.106	.47	.078	" " " No. 3.	41 100	74 360	10.0	11.2	Cr.

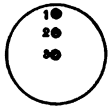

Diagram of relative position of test pieces in pins. Pieces were turned to ¾in. diameter, and drillings for analysis were taken from them after breaking. Tests A were all from the same pin, and tests B were all from a second pin.

TABLE No. 2.—Tests of Acid Bessemer Steel Pins.

Nomenclature for Fractures.—C., Cup; Gr., Granular; S., Silky.

Cast.	Test From	Physical Test. Stress. Pounds per square inch. Elastic.	Ultimate.	Percentage of strain. Elongation in 8 ins.	Reduction of area.	Nature of fracture.	Remarks.
A	6¾" rolled pin.	36 510	62 910	20.0	17.7	Gr.	Not treated.
"	" "	37 960	58 320	24.0	59.4	S. A.	Annealed in lime.
B	6¾" "	37 800	60 320	13.2	11.3	Gr.	Not treated.
"	" "	38 080	60 250	28.0	61.1	S. ½ C.	Annealed in lime.
C	" "	38 100	57 480	8.7	15.0	Gr.	Not treated.
"	" "	41 180	62 800	26.2	62.2	S. C.	Annealed in lime.
D	" "	38 900	53 120	8.0	8.7	Gr.	Not treated.
"	" "	39 190	62 160	26.0	59.3	S. A.	Annealed in lime.

The above pins were made from an ordinary quality of steel, the chemical composition of which is unknown.

The first test in each group was cut from the pin as it was finished at the rolls; the second test was cut from the same pin and annealed in lime. The appearance of the fractures of annealed test pieces was excellent.

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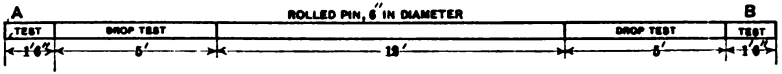
TABLE No. 3.—Tests of Acid Bessemer Steel Pin.



Nomenclature for Fractures.—A., Angular; C., Cup; Cs., Coarse; Fi., Fine; Gr., Granular; I., Irregular; S., Silky.

Chemical Test.				Test From	Physical Test.					Remarks.
Percentage of elements.					Stress. Pounds per square inch.		Percentage of strain.		Nature of fracture.	
C.	P.	Mn.	S.		Elastic.	Ultimate.	Elongation in 8 ins.	Reduction of area.		
.11	.103	.60	.061	End B, No. 1	39 710	62 780	27.0	38.8	S. ½ C.	Annealed in lime.
.11	.117	.60	.062	2	38 450	63 540	25.5	40.3	S. I.	" " "
.11	.110	.56	.060	(c) 3	35 020	59 460	13.8	12.2	Gr	Not treated.
.10	.106	.61	.058	4	35 500	62 500	15.3	13.4	"	" "
.11	.120	.60	.062	(c) 5	32 850	56 400	11.3	8.8	"	" "
.10	.106	.62	.067	End A, No. 1	41 000	60 100	30.0	65.7	S. ¾ C.	Annealed in lime.
.10	.100	.60	.055	2	40 110	59 330	30.0	66.8	S. ½ C.	" " "
.10	.104	.59	.055	3	31 520	56 850	29.5	49.3	Cs. Ck. I.	Not treated.
.10	.107	.60	.057	4	33 160	57 360	28.0	42.0	" " "	" "
.10	.114	.60	.055	5	29 840	55 300	30.0	45.5	" " "	" "

Above tests were 1 in. diameter.

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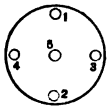
BROKEN TESTS *c* AND *e* FORGED TO ¾-IN. SQUARES.

.11	.110	.56	.06	End B, No. 3.....	43 680	67 040	18.7	33.8	Fl. S. A.	Tests not annealed, but left with hammer temper.
.11	.120	.60	.062	5.....	45 600	67 440	21.2	40.7	Fl. S T.	

DROP TESTS.—WEIGHT OF DROP, 1 640 LBS ; SUPPORTS, 3 FT. APART; TEST PIECE REVERSED AFTER EACH BLOW.

	END A.				END B.			
Number of blow.....	1	2	3	4	1	2	3	4
Fall.....	25 ft.	25 ft.	25 ft.	25 ft.	15 ft.	20 ft.	25 ft.	29 ft.
Deflection.....	2 ins.	⅛ in.	2 ins.	Broken.	1¼ ins.	¾ in.	1½ ins.	Broken.

The 6-in. round was rolled from an 8¼ x 8¼-in. bloom, which was rolled from a 15 x 18-in. ingot. The round had a short curve in the end B where the bar had followed the roll. An attempt was made to straighten this in a gag press, and at the first blow of the plunger the bar broke short off 1 ft. 6 ins from the end B.



The position of the test pieces in relation to each other and to the pin is shown in the diagram. The analyses were made from drillings taken from the broken test pieces.

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TABLE No. 4.—Tests from Basic Open-Hearth Pins.

Nomenclature for Fractures.—A., Angular; C., Cup; Ck., Cokey; Gr., Granular; S., Silky.

Cast.	Chemical Tests. Percentage of elements. C.	P.	Mn.	Test From	Physical Test. Stress, pounds per square inch. Elastic.	Ultimate.	Percentage of strain. Elongation in 8 ins.	Reduction of area.	Nature of Fracture.	Remarks.	Size of test.
A	.29	.031	.63	6¼-in. forged pin.	37 620	67 840	25.0	36.7	Ck. C.	Not treated.....	⅝ in. diameter.
"	Cast a	nalys	is	" " ..	40 640	67 300	18.0	30.2	" ⅓ C.	"	1⅛ in. "
"	.....	.....	.....	" " ..	40 000	72 200	18.0	28.4	Gr.	" .....2 x ⅛ in.	
"	.....	.....	.....	" " ..	36 760	68 520	22.7	49.0	S. ⅓ C.	Annealed in lime.....	⅝ in. diameter.
The above tests all from the same pin, forged from an 8⅛ x 8⅛-in. bloom.											
B	.29	.032	.52	6¼-in forged pin...	39 510	67 350	20.0	30.0	50% Gr.	Not treated.....	1 in. diameter.
"	Cast a	nalys	is ...	" " ..	35 790	64 620	26.2	49.9	S. ⅓ C.	Annealed in sand.....	1⅛ in. "
"	.....	.....	.....	" " ..	39 530	66 150	26.2	41.9	S. A	" "	" "
C	.29	.032	.53	6¼-in. forged pin..	39 220	68 560	20.0	27.7	75% Gr.	Not treated.....	1 in. "
"	Cast a	nalys	is	" " ..	38 160	65 270	26.5	43.6	S. A.	Annealed in sand.....	1⅛ in. "
"	.....	.....	.....	" " ..	39 490	66 280	26.5	48.0	S. ⅓C.	" "	" "

Tests marked, "not treated," were cut from pins as finished by the hammer. In casts B and C, the second and third tests in each group were cut from pins which, after being forged, were reheated to a red heat and buried in dry sand for 24 hours.

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DISCUSSION.

Mr. Metcalf. WILLIAM METCALF, Past-President Am. Soc. C. E.—The paper is of great value, as it gives important information which engineers cannot obtain otherwise, unless they are familiar with the practice of steel manufacture. The dimensions and right use of the rolls and the hammers are important matters in making pins and in all forging. They may be large enough, but the work done may be too light to penetrate the mass of the metal as it should. There is one simple means by which the inspecting engineer can always determine whether the work has penetrated the mass or not. If the finished bar or forging has concave ends, the outside material has simply been drawn over that inside, the work has not penetrated the mass, and a bad state of affairs results, as mentioned in the paper, from too little rolling or too little hammering. When a bar comes out practically straight or rounded on the end, the work done upon it has been sufficient to penetrate the softer material in the center. When the ends are convex, the work is complete so far as the forging or rolling goes.

The weight of the hammer is very important. The rule in ordinary practice is that a 1 000-lb. to 1 200-lb. hammer should never be used on a section larger than 1½ ins. square. From 1½ to 3 ins. a 2 000-lb. hammer will do very well. Between 2 and 3 ins. a 2 000-lb. to 2 500-lb. hammer ought to be employed, while for such work as bridge pins up to 6 ins. in diameter, a hammer weighing at least 3 tons is necessary to do the work properly.

Bursting is a more serious danger than piping in hammering round pins. If a piece is hammered too heavily with flat dies, the center is bound to burst. This effect of radial work is emphasized and illustrated in the Mannesmann process of heavy radial rolling, by which means a hollow tube is produced from a solid billet. If the dies are shaped so as to offer three supporting surfaces to the bar, or if the piece is first knocked into a rough octagon and then swaged in grooved dies, it is extremely difficult to burst a pin. Engineers should refuse to accept a pin forged under flat dies, since it requires the highest degree of skill to produce a sound round section in this way.

The author specifies that annealing should be done by heating the bar to a dark orange red, usually called cherry red, and leaving it in the furnace for 24 hours. This is a dangerous practice for the reason that it is not always sure the furnace will remain at its final heat for this time. Radiation may make the furnace still hotter, when the long-continued heating with slow cooling will cause a separation of the carbon from the steel, produce a granular condition of the metal and weaken the steel greatly. It is preferable and all-important in

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the annealing to heat the bar a little higher, to a fair, medium orange color, and the moment it is heated through, to bury it in either warm ashes or sand, where it will cool slowly. After a pin is forged, if it be heated thoroughly to a fair, medium orange color, about 1 100° Fahr., all of the coarse grain in the center will become as fine as the finest part of the steel. If the heat is uniform, the grain will be uniform. If a piece be taken from the fire as soon as it is heated through uniformly to a medium orange color and cooled slowly, it will have a fine grain and be free from all strains, and the carbon will remain in solution, which is its best condition. Long-continued heating of any steel will cause carbon to separate out in the mass into a graphitic form, and in that condition it is of no more use than so much sand. Mr. Metcalf.

Silicon up to 0.2% may be allowed because of its value in reducing blow-holes; but, if steel be sound, the lower the silicon the better, as its tendency is to cause brittleness. There need never be any good reason for more than 0.2% silicon, and there ought not to be a necessity for so much. The maximum limit of manganese should be 0.5 and the minimum as low as possible, because manganese is useful only as a reducer of oxides; it is not otherwise a valuable constituent in steel. In a specification for crank pin steel for locomotives, there occur the same chemical limits as those in the specification given in the paper, except that a maximum limit is provided for manganese, but the physical specification is that the pins shall have not less than 85 000 lbs. tensile strength, nor less than 15% extension in 8 ins. There is no difficulty in making open-hearth steel having 0.6 carbon with 95 000 lbs. tensile strength and 15% extension. Such steel should be annealed to give it uniformity. If crank pins, which have to withstand the severe strains of locomotive service, are better at this higher specification, it is hard to see why bridge pins, which are not subjected to such severe strains, would not be better if made under similar specifications.

Boring the pins in the manner specified by Past-President Morison will either remove the burst or enable the defects to be seen. The removal of the core of the pins in this way does not weaken the steel.

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CORRESPONDENCE.

Mr. DeForest. GEO. T. DEFOREST, Assoc. M. Am. Soc. C. E.—The writer fully agrees with the author as to the unreliability of reports on shape tests showing the condition of steel in pins, and as the trouble seems to be caused by the manipulation of the material he does not think tests of pin material particularly valuable except in that they show quality rather than condition of material used.

A few years ago the writer had an experience with some 7 and 10-in. pins. The material was made under a mild steel specification (open-hearth or Bessemer) with an ultimate strength of 60 000 to 68 000 lbs.; elastic limit, 37 500 lbs.; and elongation, 22%; steel in pins to be sound. No chemical analysis was required. These pins were, of course, forged, and while the writer was not connected with the inspection at the mill, he knows they were made in a mill where there are all the best facilities, both for heating and forging. Apparently a great deal of work was put on them as they were nearly perfect as to roundness.

They came to the shop, after inspection at the mill, cut by saw to approximate lengths. When pins were turned up and cut to length in the lathe it was noticed that the centers were badly segregated in about one-quarter of the 7-in. pins, while of the 10-in. pins five out of ten were in this condition, in some cases the segregation apparent to the eye being 1 in. in diameter, and in the one pin experimented on by drilling it seemed to run the entire length of the pin.

This seems to show the utter unreliability for showing condition of material in pins of the ordinary tests and inspections at the mill, and would also seem to indicate the necessity of separate treatment for each pin, namely, a very thorough surface inspection while the lathe work is going on in addition to the physical and chemical analysis, or better still, to adopt the clause in the specifications of one eminent bridge engineer that "all pins shall be drilled through their axes," which certainly ought to show the condition of the material at the center.

Mr. Morison. GEORGE S. MORISON, Past-President Am. Soc. C. E.—The principal point developed in the paper is one which is very important, but is not new. It has been long recognized that pins do not receive the work in manufacture which smaller sections do, and are, therefore, not as uniform and good a material. It has also been known that large rolled pins were generally of inferior character, and that if large pins were forged under light hammers, piping was usually found on the axes.

The writer's attention was called to the existence of piping in pins some years ago, and a little reflection showed that when piping did not

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actually exist, the same circumstances which had produced it in one pin were likely to have produced a very inferior material at the center of another pin. The case is analogous to steamship shafts where the difficulty is removed by longitudinal boring. If the same defects and strains exist on the axis of a bridge pin which exist on the axis of a large shaft, it seemed to him that the method which has successfully relieved this difficulty in shafts could be applied with advantage to bridge pins. Mr. Morison.

Accordingly he incorporated in the specifications for the Cairo Bridge, which were dated March 23d, 1887, the following clause:

"Steel for pins shall be sound and entirely free from piping. All pins more than 4 ins. in diameter shall be drilled through the axes."

The same clause was inserted in the specifications for the Sioux City and the Nebraska City Bridges, which were built about the same time, as well as in the specifications for the Riparia and Jacksonville Bridges which were built a year or two later. In the specifications for the Memphis Bridge, where some of the pins were exceptionally large, the following clause was inserted:

"Steel for pins shall be sound and entirely free from piping. All pins in the main trusses shall be annealed before they are turned and shall be drilled through the axes."

At first, manufacturers objected to this drilling, but the difficulties disappeared when the work began. The first manufacturer drilled the pin from both ends and exhibited a little flat disc which came out on the completion of the second cut, as evidence of the accuracy of his work.

The hole through the axis was used to pass a bolt through to hold washers at the ends of the pin, thus avoiding the necessity of tapping the pin for nuts. This, however, is not a necessary feature of the drilled pin, and nuts of the common form can be used.

The writer's experience with drilled pins has been such as to convince him of the importance of the practice. In one bridge the drilling revealed a great deal of piping and caused the rejection of nearly all the pins that were first made. It appeared that these pins had been forged under a light hammer which was worked with a large steam pressure on the downward stroke, and when he inquired the weight of the hammer, an equivalent weight was given to him instead of the real weight, the force of the steam being added to the weight. It is the mass of matter in the hammer and not the force of the blow which determines the penetration of the effect.

The writer believes that a proper specification would contain a provision fixing a minimum weight of hammer for each diameter of pin, but he has not yet satisfied himself as to what this weight should be; it should probably be a multiple of the cube of the diameter. In his latest specifications he has omitted the requirement for drilling on the

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Mr. Morison. axis, but has done so simply because it seemed to him a feature which was more properly shown on the plans than in the specifications. In specifications which are drawn for manufacturers who will make their own plans it should always be inserted.

On one occasion the manufacturer saw fit to drill the pins with a hollow tool which left a solid core taken directly from the center of the pin; this manufacturer has a very heavy steam hammer under which these pins were forged and no defects were revealed even in this core. The fact that such a core can be made without serious difficulty, the manufacturer in this case doing it voluntarily without request, provides a method for testing pins in the most rigid way.

Mr. Cunningham. A. C. CUNNINGHAM, M. Am. Soc. C. E.—The correspondence on this subject illustrates in a forcible manner the variation in results due to an ordinary specification and inspection, and a special specification rigidly enforced.

The specification quoted by Mr. De Forest admits any of the general kinds of steel, without chemical limitations, with a moderate elongation and no reduction of area. It is the kind of a specification that a manufacturer glances at but once, and after noting on the order "Use stock steel, 60 000 to 68 000," places in his files to be forgotten.

Although not mentioned by Mr. Morison, his specifications contain, besides the special requirements for pins quoted, chemical limitations for the steel, with full, complete and rigid physical requirements. This is the kind of a specification that the manufacturer keeps in his pocket for constant and ready reference.

Under the first specification it will be seen that from one-quarter to one-half of the pins sent to the bridge shop show visible defects, and what their chemical defects may have been is a matter of conjecture. Under the second set of specifications there were, at first, objections on the part of manufacturers, a misleading statement, and the rejection of nearly all the pins that were first made. Then follows compliance with the requirements, improvements in the requirements themselves, and finally the manufacturer voluntarily doing more than was required of him. A more forcible demonstration of the value of a good specification and rigid supervision could not well be found.

As the matter of segregation has much prominence in this paper, it may be well to describe it for the benefit of those who have not given it attention up to the present time; it is, of course, not new, but is very important.

When steel is first cast into an ingot mold it is in a uniform condition chemically, provided the carbon, manganese, silicon, or whatever else may have been added to it when it was tapped from the furnace, has been thoroughly mixed with it. That the paper sacks of powdered charcoal, or the coarse and cold chunks of spiegeleisen, ferro-manganese or ferro-silicon that are sometimes thrown or shoveled

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haphazard into the ladle, are thoroughly mixed is an open question, and irregularities due to this cause must not be confounded with segregation. Mr. Cunningham.

The liquid and chemically uniform steel in the ingot mold begins to cool and solidify from the outside toward the axis, from the bottom towards the top, and, to a less extent, from the top downwards.

When this process of cooling and solidifying begins, some of the substances associated with the steel commence to move toward the more liquid portions, to segregate or separate out. Of most importance among the segregating compounds are those of carbon, phosphorus and sulphur. These are moving in the lines of cooling or toward the axis and upper part of the ingot. The entire amount does not move; some is retained where it was first located, and the increase from the first solidified portions to the last is gradual. The part of the ingot which sets last is probably shaped somewhat like an inverted cone. It is on the axis of the ingot and in its upper third, and in this portion will be found the greatest segregation.

The amount of segregation which takes place in an ingot depends upon its size, shape, temperature of steel when cast, rate of cooling, and amount of segregating compounds contained in the steel. The quicker the steel solidifies, the less will be the segregation. The smaller the amount of segregating compounds in the steel, the less the tendency toward segregation. Segregation is not always found and, when found is not always excessive, though cases have been noticed where phosphorus and sulphur have doubled or even more between the outside of an ingot and the axis.

The suggestions contained in the discussion of Mr. Metcalf are of such value and importance that it will be well to incorporate most of them in the specification.

The medium orange color recommended by Mr. Metcalf for annealing is the heat of recalescence, so admirably and clearly described in his new work "Steel," at which steel assumes its finest grain and retains it if heated no higher. Very high authorities state that manganese is not only a reducer of oxides in steel, but is a neutralizer of the ill effects of phosphorus and sulphur, by preventing the coarse crystalization which they tend to produce; also that it makes steel more plastic when hot, and renders it capable of standing high heats without injury. The book by H. H. Campbell, M. Am. Soc. C. E., entitled "Structural Steel," is the latest authority which inclines to these views of manganese.

It is a matter of history that steel was not a commercial success until manganese was incorporated in it. It is also a matter of history that in the early manufacture of steel, manganese was used in excess to cover up bad practice and bad chemistry. The enemies of manganese call it a "cloak for sins," but they also want a little in steel;

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Mr. Cunningham.

its friends might as consistently call it a poultice for evils, but at the same time they do not want too much.

Manganese makes steel brittle, especially when quenched or hardened, but the upper limit in this specification is not an excess for the grade of steel specified, and this steel is to be annealed and not hardened. The lower limit is needed to insure the qualities which it has been mentioned that manganese imparts, and a good steel maker working under this specification should produce a steel with from 0.55% to 0.6% of manganese. Steel rails which are a much harder and a much poorer kind of steel than this specification provides for, contain from 0.8% to 1% of manganese. Manganese does not segregate, and even if the upper limit of the specification is reached, it will not exist in any greater amount in any portion of the steel.

The crank-pin steel mentioned by Mr. Metcalf might make excellent bridge pins if the pins were forged, annealed and worked with the same care as crank pins. The bridge pin steel, however, would not do for crank pins, which must have great rigidity to keep the size within bounds and to resist the great bending stress due to their being fixed at only one end; resistance to abrasion is also an important quality in a crank-pin.

Whether the expense of boring an axial hole through the entire lot of pins for a bridge to determine their solidity is warranted or not is open to question. The piping which may form when an ingot is cast can and should be all cut away, and it would be a bad case, indeed, in which it extended more than one-third the way through an ingot. Moreover, these pipes or the bursts due to bad forging or segregated steel, which are sometimes mistaken for pipes, are not necessarily on the axis.

If there is a pipe proper in a pin or an extensive burst, the machine finishing of the end in a lathe will reveal it. If a burst does not reach either end of the pin, an axial hole may pass within less than a quarter of an inch of it without revealing it.

The following revised specification for bridge pins is submitted :

Specification for Bridge Pins.

1.—Bridge pins shall be made of open-hearth steel.

Note.—Open-hearth steel is specified on account of its known superiority to Bessemer.

2.—If made of acid steel, the cast analysis shall show not more than 0.06% of phosphorus, nor more than 0.03% of sulphur. If made of basic steel the cast analysis shall show not more than 0.03% of phosphorus, nor more than 0.03% of sulphur.

Note.—Acid and basic steel are placed in competition, and as the basic process is essentially a dephosphorizing one, the lower phosphorus is specified in order that there may be competition; and if

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basic steel is used, the purchaser may have the benefit of the lower phosphorus. The phosphorus and sulphur limits for both steels are made such that if segregation takes place and is not discovered, it is not likely to be dangerous. Mr. Cunningham.

3.—The manganese for either steel shall not be less than 0.5%, nor more than 0.65 per cent.

NOTE.—These limits are higher than open-hearth practice for plates and angles. The pins will be heated several times during manufacture, perhaps unevenly and perhaps very hot, and these limits, which are well within the point of safety, are specified in order that the pins may stand such treatment with the least injury.

4.—The smallest diameter of the final bloom or ingot to be made into pins shall be at least 50% greater than the diameter of the pin.

NOTE.—This smallest size of bloom or ingot is specified in order to secure not less than a certain amount of work in finishing the pin.

5.—Whether rolled or forged the crop ends of all pins shall have a convex end.

NOTE.—The convex end for crops is to secure sufficiently heavy hammer or rolls to have the work penetrate the entire pin.

6.—Forged pins shall be made with concave dies of a form satisfactory to the engineer.

NOTE.—Concave dies of the correct form are least likely to burst the steel.

7.—All pins shall be stamped with the cast number of the steel from which they are made, and shall be numbered consecutively from the bottom of the ingot to the top.

NOTE.—The consecutive numbering is in order that pins may be readily selected for segregation tests.

8.—After pins have been manufactured to diameter they shall be carefully and uniformly heated to a medium orange color in a closed furnace out of contact with the fuel, after which they shall be buried in warm dry sand or ashes until cool.

NOTE.—Pins are annealed to remove any initial stress that may be set up, and to give them a uniform, fine structure.

9.—Tests cut from near the surface of annealed pins and of a sectional area of not less than ¼ sq. in. shall have a maximum strength of not less than 60 000 lbs., nor more than 70 000 lbs. per square inch, an elastic limit of not less than one-half the maximum strength, an elongation in 8 ins. of not less than 20%, and a reduction of area at fracture of not less than 40 per cent.

Tests cut from near the center of pins shall comply with the above requirements, except that a fall of 5 000 lbs. in maximum strength, and a fall of 5% in reduction of area will be allowed.

NOTE.—This grade can be readily made in acid or basic steel. The variation between the surface and center is allowed on account of

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Mr. Cunningham. the natural falling off in physical properties as the latter is approached.

10.—The fractures of all tension tests shall be free from granulation.

NOTE.—Granulation indicates poor work, poor heating, or poor annealing.

11.—Tests cut as before specified shall bend, at the normal temperature, 90° around a diameter equal to the thickness of the test, and between a dark red and a light yellow heat, 180° flat on themselves, all without rupture on the outside of the bent portion.

NOTE.—These bends are a protection against cold and hot or red short steel.

12. - Analyses of drillings taken from near the surface of pins shall not exceed the limits of the cast analysis by more than 10%, and, when taken from near the axis, shall not exceed said limits by more than 50 per cent.

NOTE.—This puts a safe limit on the allowable segregation.

13.—Tests and analyses shall be made on such pins as the engineer shall designate.

NOTE.—Pins being numbered consecutively from the bottom of the ingot, the engineer will be able to select those most likely to contain segregation.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 786.

IMPROVING THE ENTRANCE TO A BAR HARBOR BY A SINGLE JETTY.

By T. W. SYMONS, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, 1896.

WITH DISCUSSION.

In devising plans for the improvement of the entrances to the tidal harbors of the United States, the general system adopted has been to confine the outflowing and inflowing waters between parallel or converging jetties so located and arranged as to direct the waters upon the bar in a fixed position. Where single jetties have been planned, as at the mouth of the Columbia and San Diego, it has been because the other jetty was replaced by headland or permanent shore. The author's experience in charge of the improvement of a number of harbors on the Pacific Coast has convinced him that there are places where the double jetties are unnecessary and where a single jetty, if properly placed, will, in conjunction with natural tendencies, compel the entrance channel to adopt a fixed and permanent location and practically accomplish all that could be expected of two jetties, and at half, and sometimes less than half, the necessary cost of a pair.

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Having to prepare plans for the improvement of the entrance to Grays Harbor in the State of Washington, he became satisfied from a study of the situation, the currents, the tides, the volume of flowing water and the channel movements, that here was a place where a single jetty was applicable. The improvement of the harbor was so planned, and the plan is presented to the Society in this paper, with the facts and reasons which led him to adopt it.

General Description.—It is considered a matter of importance if in certain places it is reasonable to expect to effect a marked and permanent improvement in a harbor entrance by a single jetty. There are places where improvement is greatly needed, where the cost of double jetties would be prohibitory, but which could well stand the expense of a single jetty. It is a satisfaction in making a plan for a single jetty to know that if it be unsuccessful, it will form an essential part of a system of double jetties or other combination of structures necessary to secure the requisite control of the flowing waters.

The entrance to Grays Harbor from the Pacific Ocean lies in latitude 46° 55′ north, being 45 miles north of the mouth of the Columbia River.

The harbor has a length from east to west of 17 miles, and from north to south a maximum breadth of 14 miles. There are a number of rivers tributary to the harbor, the principal being the Chehalis, Satsop, Wynooche, Wishkah, Hoquiam and Humptulips, all of which come from the east and north, and drain one of the richest and most magnificently timbered sections of the State of Washington. There are two main channels crossing the harbor from east to west, of which the north channel is the principal one and the one undergoing improvement at its upper end. A large part of the harbor is occupied by tide flats, bare at low water. The total tidal area of the harbor is 96.8 square miles. At low tide the area covered by water is 30.6 square miles, or less than one-third of the total area. The average range of the tide is 8.4 ft., with a maximum range of 12.9 ft. The low area consists of a number of channels running through and between mud and sand flats. The banks of these channels are quite steep, so that the water does not begin to overflow the flats until it has risen about 2 ft. above mean low water. At about 5.2 ft. above mean low water, the flats are completely covered. Between this stage and mean high water, there is no considerable increase in water area.

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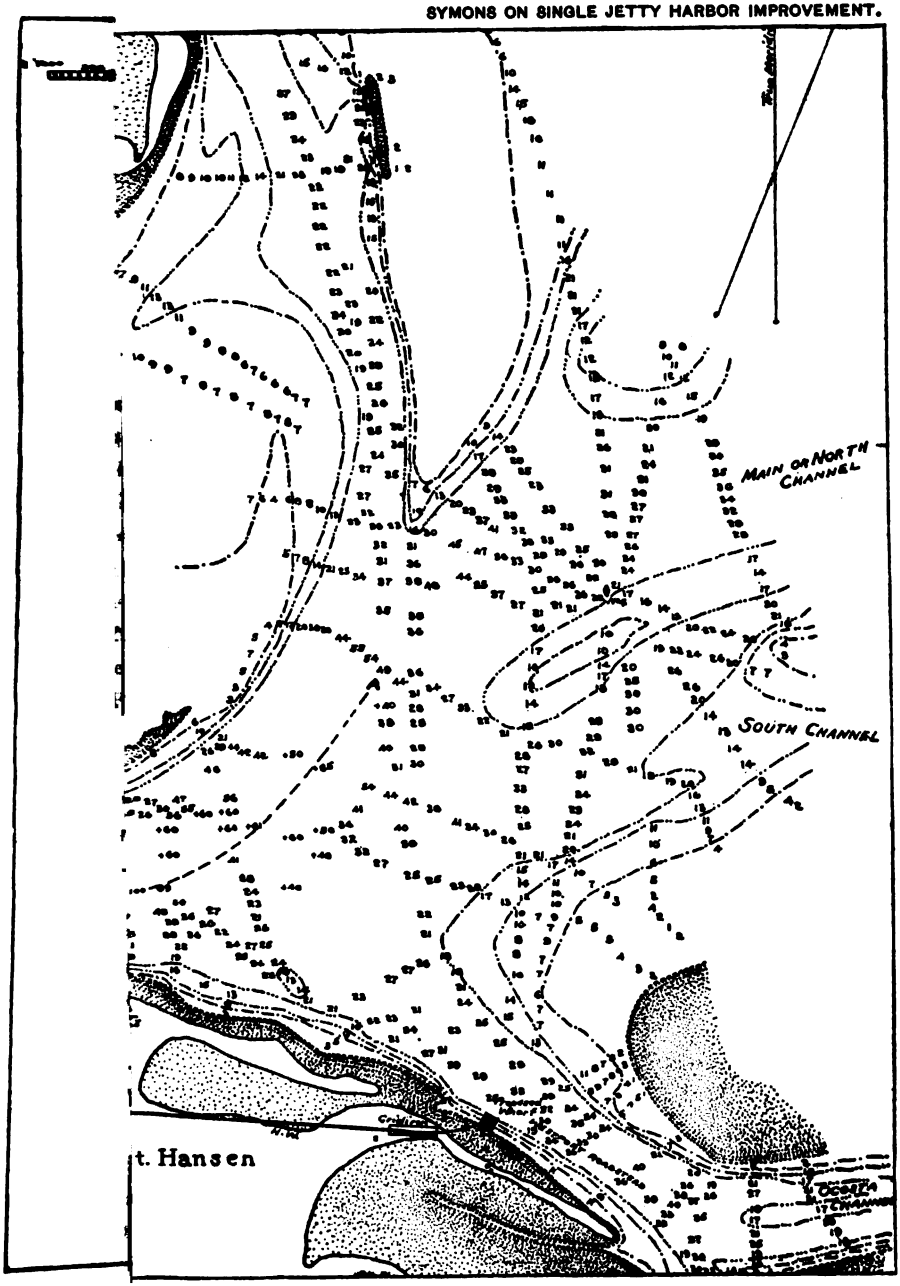

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From the data available, it is estimated that the volume of water displaced during a mean tide of 8.4 ft. is 16 000 000 000 cu. ft. Assuming the tide to run out for six hours, this gives an outflow of 748 000 cu. ft. per second. The volume of water displaced during a maximum tide amounts to 24 000 000 000 cu. ft., which, assuming the run-out to last six hours, makes a discharge of about 1 100 000 cu. ft. per second.

The harbor throat lies between two low, sandy peninsulas terminating on the north of the entrance at Point Brown, and on the south at Point Hanson. The distance between high-water lines on these two points is 12 500 ft. The survey of Grays Harbor entrance was made in October, 1894. The results of the survey are shown on the accompanying map, Plate IV, upon which is indicated the location of the jetty proposed for the improvement of the harbor entrance. The soundings on the map are in feet, and are referred to the mean of the lower low waters.

The survey shows maximum depths of 100 ft. in the harbor throat. A single broad waterway extends for more than 2 miles from the harbor throat out to sea, with depths gradually diminishing from 100 ft. to 30 ft. Between this deep waterway and the open sea lies the usual bar, convex to the sea, and connected with the spits which jut out from Points Brown and Hanson. The spit from Point Brown is the North Spit, and that from Point Hanson is South or Trustee Spit. Across this bar there is no well-defined, decided, permanent channel. The best water was found at the time of the survey to be in a line southwesterly from Buoy No. 2. This was the channel used by tugs and boats entering the harbor, and was marked by a mid-channel buoy. The survey shows that fully as good a channel existed to the east of this, running more nearly due south. There is also shown nearly as good a channel to the northwest, and also a fairly good one nearly due west.

The governing depth on the bar in the south and southwest channels shown was 13 ft. at mean lower low water. In the northwest channel it was 12 ft.

The general average distance between the inner and outer 18-ft. curves is ½ mile. The region between the two curves was found to be irregular, with lumps and depressions.

On the north of the entrance the extreme limit of Point Brown is a ribbon of sand partially encircling the main higher land, and separated

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from it by a tidal lagoon called Johns Bay. The same formation, but less marked, exhibits itself on the south. Maps of the entrance made at different periods show marked changes in the formation about these limiting points.

There seems to be a definite cycle of changes in the end of the north spit abreast of the harbor throat. First, there is an island near the deep water shown on the 1862 map as Eld Island, see Fig. 1. This island gradually works to the north, and finally attaches itself to and encircles Point Brown, as shown on the 1891 map, Fig. 2. This ribbon gradually wears away, as shown on the 1894 map, and, as this proceeds, the island begins to form near the deep channel toward Point Hanson. This island is now in progress of formation, although it was not possible at the time of the recent survey to get any definite soundings on it, to show it in detail. It is, however, indicated in position on the map.

The changes about Point Hanson are less in extent than those about Point Brown. The 1862 map shows an extension of the Point Hanson sands farther to the north than they appear at any subsequent period.

Deep water in the harbor throat lies next to Point Hanson, and occupies about one-third of the width between it and Point Brown. The other two-thirds is occupied by the changeable sandspit above described, which is partly bare at extreme low tide. In the harbor throat the flowing waters have gouged a deep hole. The greatest depth found at the time of the survey was 100 ft., but depths as great as 106 ft. are shown on some of the Coast Survey charts.

A cross-section of the harbor throat is shown in Fig. 3, and upon the same sketch are shown the longitudinal profiles along the entrance channels and the longitudinal profile of the proposed jetty.

The cross-sectional areas of the channel between Points Brown and Hanson are as follows:

	Square feet.
At mean high tide	293 760
At mean tide	245 960
At mean low tide	200 560

This would give, in an ordinary tide, an average velocity through the harbor throat of 3.04 ft. per second, or about 2 miles per hour. In a maximum tide this average velocity would be increased to 4.47 ft.

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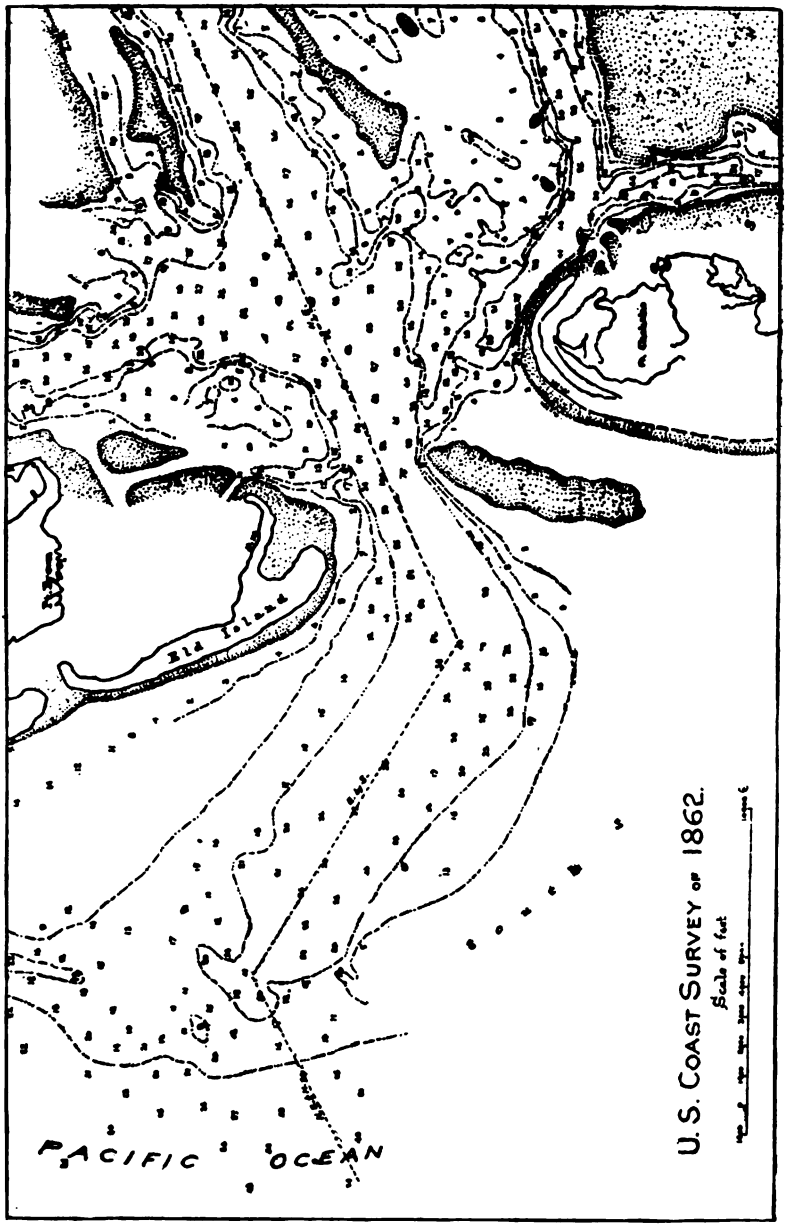

FIG. 1.

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per second, or about 3 miles per hour, with extreme velocities of 6 to 7 miles per hour.

Points Brown and Hanson are the terminations of low peninsulas which separate Grays Harbor from the ocean. The Point Brown peninsula is about 7 miles long, and the Point Hanson peninsula about 4 miles long. Both average 1 mile in width and both are covered with a dense jungle of timber to within about 1½ miles of their extreme ends.

In front of the entrance to Grays Harbor the 100-fathom curve lies 32½ statute miles from the general shore line. The 10-fathom curve lies 4½ statute miles in front of the general shore line. This gives a general slope to the ocean bed in front of the harbor of 3.2 fathoms or 19.2 ft. per statute mile. The 10-fathom curve lies but about 3 000 ft. in front of the outer 3-fathom curve of the bar. This gives a very steep slope in front of the bar of 1.4 ft. in 100 ft., a slope which is very conducive to the success of the plan of improvement proposed, as the bar will not be readily formed in advance of its present position after a deep channel is scoured across it.

The Grays Harbor bar projects beyond the general coast line in a crescent shape, as is well shown on Coast Survey charts Nos. 6 100 and 6 400. The material of the bar is fine gray sand. Directly opposite the entrance the outer 3-fathom curve is 4 miles beyond the middle of the harbor throat. The general distance between the inner and outer 3-fathom curves varies greatly, but it may be said to be approximately ½ mile in the vicinity of the bar channel.

The best bar channel varies in depth from 12 to 20 ft., and in direction from nearly south to northwest. As nearly as can be ascertained there is a great cyclic change in the principal bar channel, occurring in about 35 to 40 years, and a lesser one occurring at short and irregular intervals. The greater movement seems to follow or be coexistent with the building up and movement of the sand island off the southern end of Point Brown. When the island is well formed, the best entrance channel is to the northwest; and as the island travels north, the channel travels south.

The entrance channel is at its best when leading out to the southwest. From this position it swings gradually around to the south, until it reaches a direction that is nearly due south. As this movement takes place the channel becomes deeper, but narrow and crooked,

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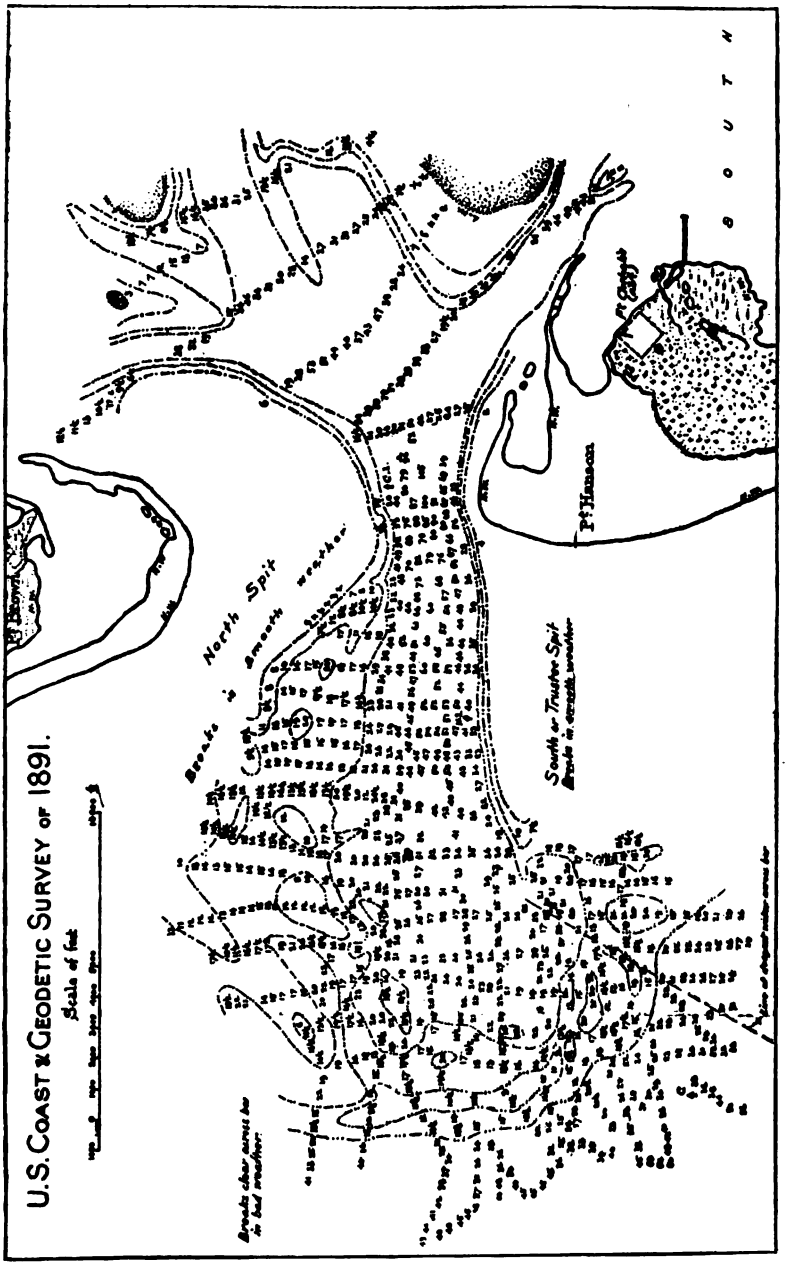

FIG. 2.

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and immediately following this it breaks out to the west and commences to swing around to the southward again. The smaller movement, from west around to south, occurs many times more than the greater from northwest around to south. This constant, gradual movement of the channel from north to south has an important bearing upon the plan of improvement recommended. It is this movement or tendency which it is believed will make a single jetty interposed in its way effective in maintaining a permanent deep bar channel.

During the winter, when the southwest storms prevail, the water is deeper in the entrance bar channel than during the summer. This is attributed to the heavy freshets more than counteracting the effects of the storm to close the bar. The bar channel closes to some extent during the summer, when the milder northwest winds prevail, during which time the sands of the higher spits and beaches are dry, and the ebb and flow of the tide is small, and there are no freshets.

Littoral Current.—In the vicinity of Grays Harbor there is a littoral current, the general resultant of which tends to the north. This general current to the northward is ordinarily considered as the eddy caused by the southward flowing Japanese Gulf Stream. It is noticed all along the coast and has received the name of the Davidson Inshore Eddy Current. While the resultant littoral current is to the north, it ordinarily follows the direction of the prevailing winds in the immediate vicinity of the land, moving to the south in summer and to the north during winter. In intensity it is said to reach as high as 2½ miles per hour either way.

Surveys.—There have been five surveys of the Grays Harbor entrance made, covering a period of 32 years. Three were made by the Coast Survey in 1862, 1883 and 1891, and two by the United States Engineer Department, 1881 and 1894.

Charts of the surveys of 1862 and 1891 are given in Figs. 1 and 2. The survey of 1894 is shown on a larger scale in Plate IV. The bar and bar channel shown on the charts of 1891 and 1894 are similar, and unlike those shown on the 1862 chart.

In 1862 the bar channel was to the northwest of the harbor throat; in all the others it is to the southwest. The complete charts of 1891 and 1894 show the bar to be in a lumpy condition. The survey of 1862 shows but one channel, that leading to the northwest. It is quite pos-

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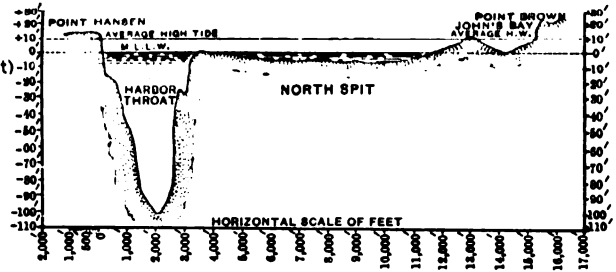
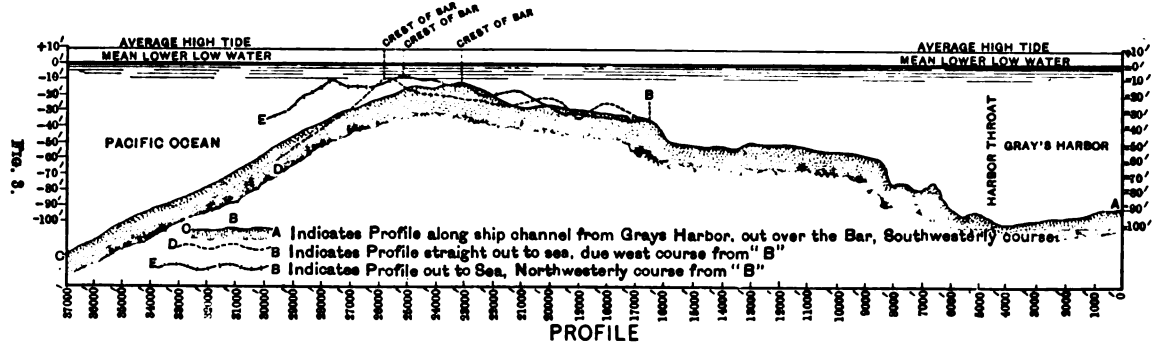
CROSS SECTION AT G H (Harbor throat)

Area of Waterway
At M.L.L.W. = 200,560 Sq. feet
At Mean of H.& L.W. = 245,960 " "
At Mean High Tide = 293,760 " "





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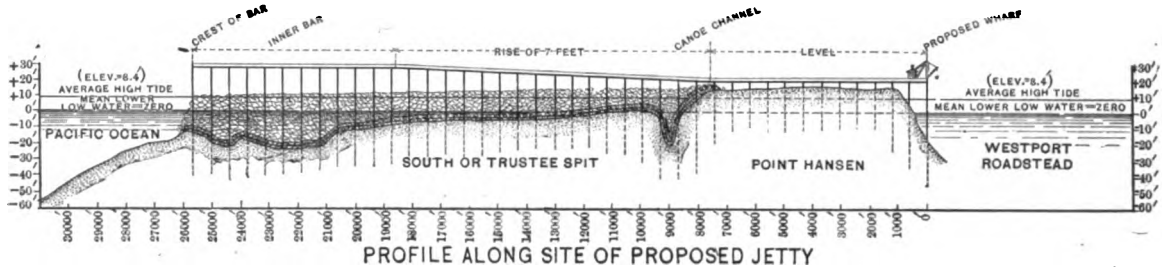

PROFILE ALONG SITE OF PROPOSED JETTY

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sible, and in the author's opinion very probable, that a good channel existed also to the southwest, although not shown on the chart. The survey was made in a sailing brig, and the soundings were apparently confined almost entirely to the waterway leading out to the northwest, which was probably the best known channel at the time. No soundings are shown to the south and southwest.

There is another condition bearing upon the question of improvement which is not fully shown on the charts. This is the existence of a channel leading to the south around and close in to Point Hanson. This is indicated on the map of 1862, and slightly on the map of 1881, and was observed, but not surveyed, in 1894. It is marked on the map as Canoe Channel from the fact that it is used by fishermen and others going in and out in small boats, in good weather, when the sea is smooth.

Plan of Improvement.—The plan proposed for the improvement of the entrance to Grays Harbor is based upon the theory of controlling the ebbing and flooding waters to a sufficient extent to concentrate and direct upon the bar in a fixed location a much greater portion of these waters than would naturally go there, thereby scouring a channel across the bar of depth ample for all purposes of navigation and permanent in position.

The large tidal volume and discharge from Grays Harbor furnish the means, if properly directed, to secure a bar channel of satisfactory depth. The ordinary mean tidal discharge is estimated at 748 000 cu. ft. per second. Its tremendous scouring effect is shown at the harbor throat, where, as long as the harbor has been known, depths of 100 ft. have been maintained.

It is recognized that in order to produce a satisfactory channel it is necessary to add to a natural channel but a limited proportion of the ebbing and flooding waters now going elsewhere, provided these waters can be kept flowing there permanently. In other words, it is by no means necessary, as it is not desirable, to direct all the ebbing and flooding waters into a single narrow bar channel. It is also recognized that the safety of any controlling works demands that but a small proportion of the flowing waters be fully controlled by artificial means.

Based upon the ordinarily accepted theories of harbor improvement, as shown by the plans adopted for various places in the United

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States, the plan for Grays Harbor would be to confine the outflowing and inflowing waters between two jetties more or less parallel and converging upon the bar at a sufficient distance apart to concentrate and direct the waters in a single bar channel. Recent experiences and consideration of the subject convince the author, however, that at Grays Harbor this ordinary plan can be deviated from, and that one jetty, if properly located, will accomplish results fully as good as two, at a vastly less cost, and without any undue or destructive strain upon the structure from the developed currents.

At Coos Bay, Ore., the one jetty that has been built has caused a bar channel to be developed and maintained fully equal to that which it was hoped to accomplish with two jetties. There the jetty has been interposed against the slow, constant movement of the bar channel to the northward. The plan proposed for Grays Harbor is based largely upon the results obtained and studies made at Coos Bay. At the mouth of the Columbia the single jetty has been eminently successful. Here there is a rocky headland on the north, and the situation is apparently quite different. In reality, however, it is much more similar than it appears at first sight, for the limiting of the waters to the north of the bar channel is due, not to the Cape Disappointment headland, but to Peacock Spit and its tailing sands. There are many points of resemblance in the Grays Harbor problem to that at the Columbia River.

The plan proposed for the improvement of Grays Harbor entrance consists of a single high-tide, rubblestone jetty, founded on Point Hanson, and directed out to sea in a direction nearly due west.

The receiving wharf is located on the south channel inside the harbor sufficiently far to enable stone scows, etc., to be unloaded in safety. The jetty tramway approach thence extends nearly due west 7 300 ft. across the head of Point Hanson. The jetty commencing at the high-tide line also extends nearly due west 10 900 ft. along the South or Trustee Spit at a distance of about 1 000 ft. from its outer steep slope, cutting off the inner or Canoe Channel. Leaving Trustee Spit, it curves to the southwest for 7 300 ft., reaching the crest of the bar at about the present location of the mid-channel bar buoy. Here it stops.

With this location, the water flowing to the southward through the inside coast channel over Trustee Spit, and beyond this to the end of

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the jetty, would be stopped from flowing in its present direction, and compelled to go in and out to the north of the jetty. It is believed that these deflected waters, added to the natural waters of the desired channel location, will give an improved bar channel, sufficient for all purposes of navigation.

With the jetty in this location, the slow cyclic movement of the bar channel would be stopped when the latter reaches its best and most satisfactory position, and it is believed that the jetty would maintain it in this position. The jetty would prevent the channel from moving toward the south, and there would be no occasion for it to break out to the west or northwest.

It is this interposition against the gradual movement of the channel that has made the Coos Bay and Columbia River jetties so successful. The same conditions which at Coos Bay have rendered the north jetty effective in maintaining a deep and permanent channel over the bar exist at Grays Harbor, but are reversed. At Coos Bay the bar channel habitually broke out to the west and gradually worked around to the north, getting finally into an awkward trough channel nearly parallel with the coast. The jetty as built is interposed in the way of this gradual channel movement to the northward, and it is to this fact that its good results in giving a permanent location to the bar channel are attributed.

At Grays Harbor the bar channel habitually breaks out to the west and gradually works around to the south, getting finally into an awkward trough channel nearly parallel with the coast. The proposed jetty is located to interpose itself in the way of this gradual channel movement to the southward with the hope and belief that the same resultant action which has been found at Coos Bay will be found at Grays Harbor, with the outcome of giving a deep, permanent, well-directed bar channel.

The situation at Grays Harbor in regard to exposure, latitude, bar changes, and volume of tide water is much more analogous to that at the mouth of the Columbia River than it is to that at Coos Bay. It is in fact very similar to the Columbia. At the Columbia a single south jetty has been projected and constructed, and is eminently successful in giving a deep channel in a permanent location, and this notwithstanding the fact that the entrance between the end of the jetty and Cape Disappointment is over 3 miles in width.

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The exposed end of the jetty proposed for Grays Harbor would be nearly head on to the worst storms, those from the southwest, an important point in maintaining its stability. The slightly curved direction given to the jetty trace, convex to the channel, will give the channel whatever advantage there is due to the tendency of tidal flows to follow a convex curve. The channel will also have the direction and location most convenient and desirable for the commerce centering in the harbor.

It was suggested by one in authority that the system of double jetties should be adhered to, the jetties being built about 6 000 ft. apart, protected by groins 500 ft. or more in length, leaving a clear waterway about 4 500 ft. wide. This suggestion was carefully considered, but for the reasons here given was not adopted.

Nowhere on the Pacific Coast, nor on the Atlantic Coast, nor, so far as the author is aware, in the world, has the attempt been made to control between parallel jetties a tidal flow of anywhere near the magnitude of that at Grays Harbor, and the author would view with great apprehension the attempt to build the jetties to so control the tidal flow. With an average tidal flow and the rivers tributary to the bay in their ordinary condition there would be, in a channel 4 500 ft. wide and an average of 30 ft. deep at low tide, a mean tidal velocity of 4.9 ft. per second, or 3.3 miles per hour. In this case the maximum velocity would be about 6 miles per hour. With a maximum tidal run-out in such a channel there would be a mean tidal velocity of 7.2 ft. per second, or 4.9 miles per hour, and a maximum velocity of about 9 miles per hour. This is without taking into consideration the rivers tributary to the bay, which would add considerably to the volume and velocity of the current if they were in a freshet stage.

Such velocities as these are, in the author's opinion, incompatible with safety when they result from the artificial contraction of a waterway by comparatively frail structures built upon shifting sands and exposed to the wave action on a very stormy coast. It must be admitted that such a great volume of water as that of Grays Harbor, moving with the stated velocities and aided by wind and waves, is a tremendous power and liable to play havoc with any works exposed to its full force. This is to be avoided, if possible, and it is with this end in view that it is proposed to control as small a portion of the flowing waters as possible consistent with attaining the desired end.

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At the harbor throat or gorge the cross-sectional area of the waterway at mid-tide is 245 960 ft., and here depths of 100 ft. are regularly maintained. With a width of 4 500 ft., this same cross-sectional area could only be obtained by an average depth throughout of 54 ft. It could, therefore, naturally be expected that between the jetties spaced as indicated, there would be depths exceeding 60 ft., which depth confined in the immediate vicinity of a rubble stone jetty founded on sand would be very threatening, even though the jetty were guarded by long and expensive groins.

At only two points on the Pacific Coast have systems of parallel jetties controlling the tidal expense in one concentrated stream been practically completed—at Wilmington and at Yaquina. At Wilmington the tidal flow is only one-one hundred and twentieth of that at Grays Harbor, and at Yaquina only one-twentieth of that at Grays Harbor.

The author's experience is such as to convince him that it would probably be found impossible to build a second jetty at Grays Harbor within 6 000 ft. of a first one by the ordinary method which has been and is being used on this coast. As this second jetty would be built out, the currents would scour a channel in front of it until the depths would become too great for the construction of a pile and timber tramway. As the seas in the locality are too rough to permit of the construction of the jetty from floating plant, there would be the necessity of devising new methods of construction, which would surely be far more expensive than those ordinarily used.

As indicative of the unusual magnitude of the work of improving Grays Harbor by a system of parallel jetties in the manner suggested, a table is given on page 123 showing the mean range of the tide, the tidal area, the mean tidal discharge, and distance apart of the proposed jetties for Grays Harbor and other harbors of the United States, where jetties have been or are being constructed, concerning which the author has been able to get information.

On the direct line from Point Brown to Point Hanson, the line upon which the greatest depth is found, the midtide section has an area of 246 000 sq. ft. The maximum depth on this line is about 100 ft., a depth entirely inadmissible in the vicinity of jetties unless the jetties be protected by very long and costly groins.

If a system of double jetties were adopted it would be necessary to provide a minimum waterway between them fully equal to 246 000

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sq. ft. Assuming an average midtide depth of 20 ft., this would require that the jetties should be spaced fully 2 miles apart, if built to high tide.

If this width were narrowed there would be a tendency to dangerous undermining of the jetties, which would increase as the width diminished. Within this necessary width of 2 miles there would undoubtedly develop one or more channels of much greater depth than 20 ft. at midtide. Such a channel might occupy any position between the jetties, and, on leaving their concentrating influence, would have

Locality.	Mean range of tide.	Tidal area.	Mean tidal discharge per second.	Distance between jetties at sea ends.	Remarks.
		Square miles.			
PACIFIC:	Feet.		Cubic feet.	Feet.	
Columbia River.....	6.2	*140	1 000 000	18 900	High tide single jetty. Width between end of jetty and Cape Disappointment.
Grays Harbor.....	8.4	96.8	748 000	6 000	Reduced to about 4 500 ft. by groins. Suggested width.
San Diego.....	3.7	25.5	97 000	4 000	High tide single jetty. Approximate width between end of jetty and Point Loma.
Humboldt Bay.....	5.5	24	100 000	2 100	High-tide jetties.
Coos Bay.....	5.5	24	100 000	1 500	" "
Yaquina Bay.....	7.0	5	38 000	900	" "
Siuslaw River.....	5.2	5.4	30 000	600	" "
Coquille River.....	4.1	3.6	15 000	600	" "
Wilmington Harbor..	4.0	2.25	6 000	700	" "
ATLANTIC:					
Galveston Harbor.....	1.1	450	250 000	7 000	High-tide jetties.
Charleston Harbor...	5.1	15	170 000	2 900	Low-tide jetties.
Cumberland Sound...	6.0	27	202 000	3 900	" "
Winyaw Bay.....	3.5	12	120 000	4 000	High-tide jetties.
St John's River.....	4.3	(?)	76 500	1 600	
Sabine Pass	1.4	120	50 000	1 700	High-tide jetties.
Newburyport.....	7.5	(?)	(?)	1 000	" "

* To Cathalmet.

almost as great latitude in choosing its direction as would be the case of a channel with but a single jetty as directrix. It is therefore believed that it is unnecessary to provide a north jetty or make provision or estimate for any other construction than the south jetty.

It is proposed that the jetty shall extend to the ordinary crest of the bar in its present location. This would give a total length for the structure of 25 500 ft. from the receiving wharf to the end. The jetty proposed is a rubble mound founded on a brush mattress, the

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whole built from a pile and timber tramway extending from the shore, the enrockment to be brought well above high tide.

The estimated cost of the work is as follows:

Plant including wharf, engines, derrick, locomotives, cars, machine shop tools, building, stone barges, lumber and brush barges, etc.	$153 955
Tramway, 25 500 ft. long	102 590
Mattress foundation, made of small spruce trees laid diagonally	61 900
Enrockment. Rock brought from the Columbia River, 441 100 tons at $1.07	471 977
Groins. Five, each 200 ft. long	40 940
Shore protection	15 000
Engineering, superintendence, office expenses, etc., 5 per cent	42 318
	$888 680
Contingencies, accidents, delays, repairs, care of property, etc., about 12½ per cent	111 320
	$1 000 000

The cost of the double jetty system applied to Grays Harbor, as suggested, would be about $2 500 000. Owing to difficulties that might be encountered in building the second jetty, and the possible necessity of greatly strengthening both jetties, it might be even more than this.

It is not to be denied or contended that when the single jetty is completed it may be found essential to its success to put in some shore protection or other minor works, to develop and render stable the best condition of the North Spit, but the expense of such works would be slight.

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DISCUSSION.

G. F. ALLARDT, Esq.—In the year 1890 the speaker was called upon by the Hawaiian government to devise a plan for deepening the bar off Honolulu Harbor. The bar is about 1¼ miles from shore, with a depth of 20 ft. on its crest at low tide, and consists of a deposit of coral sand, intermixed with scattering masses of coral more or less compacted or cemented. Across this bar it was proposed to excavate a straight channel 200 ft. in width to a depth of 30 ft. at low tide. The distance from the 30-ft. depth on the harbor side to the 30-ft. depth on the ocean side was 1 200 ft., and the necessary excavation amounted to about 70 000 cu. yds. In his report the speaker recommended that the excavation be made by means of a hydraulic dredge, and that the excavated material be utilized in reclaiming some shallow mud flats belonging to the government, and distant about three-fourths of a mile from the bar. Owing to the scarcity of competing contractors on the islands, it was suggested that the government acquire its own dredging plant, that the actual work of dredging be let out by the cubic yard, and that the plant be subsequently used by the government in deepening and extending the inner harbor. The recommendations were substantially adopted, and the legislature made an appropriation of $200 000 for the improvement. Mr. Allardt.

A contract was entered into for a first-class hydraulic dredge with all the necessary pipes, floats, etc., for handling and transporting the material. All machinery and iron work were made in San Francisco and then shipped to Honolulu, where they were fitted to the wooden hull constructed there, it having been considered hazardous to tow the completed machine across the ocean. The cost to the government of the dredging plant complete was $77 300.

Considerable delay in beginning the work was occasioned by rough weather, and when the work was commenced the contractors found it necessary, on account of the wave motion of the sea, to abandon the use of the revolving cutter and the spud mooring. A cast-iron mouthpiece was therefore substituted for the cutter, and connected with the suction pipe by a flexible pipe, and with this contrivance it was found that the coral sand could be drawn in to the full capacity of the pipe with no other means of loosening it than that furnished by the current of water sucked in by the pump. The spud mooring, which is used to such advantage in smooth water, was replaced by anchor moorings, with which it was difficult to keep the dredge exactly in the desired position, and the result was that the bottom of the cut presented a succession of hummocks and pits. The contractors, however, faithfully expended much time and money in removing and leveling off

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Mr. Allardt. the hummocks, so that upon the acceptance of the work the depth of water at no point was less than that required by the contract. On June 1st, 1893, the excavation was completed and the bar opened to commerce. In the language of the Minister of the Interior, "it was an improvement that opened our inner harbor and wharf facilities to steamers of the largest class touching here from China, Japan, Australia, San Francisco, and British Columbia."

No perceptible filling has since taken place in the dredged channel, the scouring action of the ebb tide, though very feeble at Honolulu, being evidently sufficient to prevent any considerable deposits. Since the bar has been completed, the government has utilized the dredging plant with excellent results in deepening and extending the inner harbor.

In view of the experience above set forth, the question may well arise whether the method adopted at Honolulu may not be applied advantageously to removing the bar at Grays Harbor. The relative situation in both places is much the same. At Honolulu there is a bar of coral sand, at Grays Harbor a bar of "fine gray sand," both bars being located in the open sea far from shore, and both exposed to the full sweep of the Pacific Ocean.

From careful measurements on the author's chart it is possible to make a fairly close estimate of the amount of excavation required to obtain a given width and depth of channel. Let it be assumed, for example, that a depth of 24 ft. at low tide is desired. This would give a depth of 32.4 ft. at average high tide and a depth of 36.9 ft. at the maximum high tide, far better water than is found in New York Harbor. Assume further, a bottom width of 200 ft., with side slopes of two horizontal to one vertical, then the total excavation would amount to 186 000 cu. yds. The cost of hydraulic dredging on the Pacific coast in comparatively still water ranges from 12 to 16 cents per cubic yard. On the Honolulu bar the cost ran up to 70 cents, but this price included the work of making the necessary walls and levees to retain the material on the land that was reclaimed. At Grays Harbor, however, there are no reclaimable areas within a practicable distance; probably the cheapest dumping ground would be the bottom of the sea outside of the bar, say, at a depth of 50 ft. This would require about 4 000 ft. of piping, which should be submerged in order to be beyond the action of the waves. Under these conditions, and considering the lively competition in dredging work on this coast, the cost per cubic yard should not exceed 50 cents; but even at the high price of 70 cents per cubic yard the entire cost of deepening the bar and making Grays Harbor accessible to the largest ships would amount only to $130 200.

The author estimates the cost of a single jetty at $1 000 000, and while he believes that one jetty will answer the purpose, he admits

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that a double jetty may, after all, be required to produce the desired result. He puts the cost of a double jetty at $2 500 000, or nearly twenty times the cost of excavating a channel by the dredging process. It may be contended that a dredged channel would be liable to silt up in the course of time, but this is an extremely remote and improbable contingency. A comparison of the charts of 1862 and 1894 (32 years) shows no material change in the depth over the bar, the minimum depth being 9 ft. at low tide on both charts. Grays Harbor bar, like other bars in tidal waters, was formed by the action of opposite forces from within and without, which, in the course of time, finally reached an equilibrium and resulted in the present permanent condition of the bar. If this condition be disturbed by artificial deepening, the same forces would tend to restore that equilibrium; but this action, judging from the immobility of the bottom during the past 32 years, and probably much longer, would undoubtedly be extremely slow, and many years must elapse before any considerable filling would take place. Mr. Allardt.

It seems evident, moreover, that when a channel is once cut through the bar, the main current of the outflowing water will naturally seek the deeper channel, just as the most rapid current in rivers invariably follows the lowest depressions, and instead of silting up the channel, would have a tendency to scour it still deeper. However that may be, the great difference of cost in favor of dredging as compared with the jetty system certainly warrants a more thorough study of the problem before the United States government commences active operations at Grays Harbor.

G. H. MENDELL, M. Am. Soc. C. E.—The natural conditions of tidal expense, exposure, prevalence of sand, slope of sea floor and meteorology are much the same at Grays Harbor as at the Columbia River entrance. The successful treatment of the latter leads one to expect favorable results from the application of the same principles to the development of the former. Col. Mendell.

The Columbia River entrance before and after improvement is illustrated in Figs. 4 and 5*. A single wide, deep and commodious channel, a very large accumulation of sand on Clatsop Spit and a a redistribution of sand deposits, interior and exterior, are the main changes to attract attention. There have usually been two or more ship channels over the bar, varying considerably in position and depth. Clatsop Spit has at times, in a state of nature, been bare at low water over a great part of its area and considerable portions have stood several feet above the low-tide plane. Contemporaneously a bar entrance of satisfactory dimensions was developed, succeeded by indifferent channels when the shoals were much lowered or their continuity much broken. Clatsop Spit was for the time a jetty, but as

* Two more charts of the Columbia River entrance were printed in *Transactions*, Vol. xxviii, Plate xx.

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Col. Mendell.

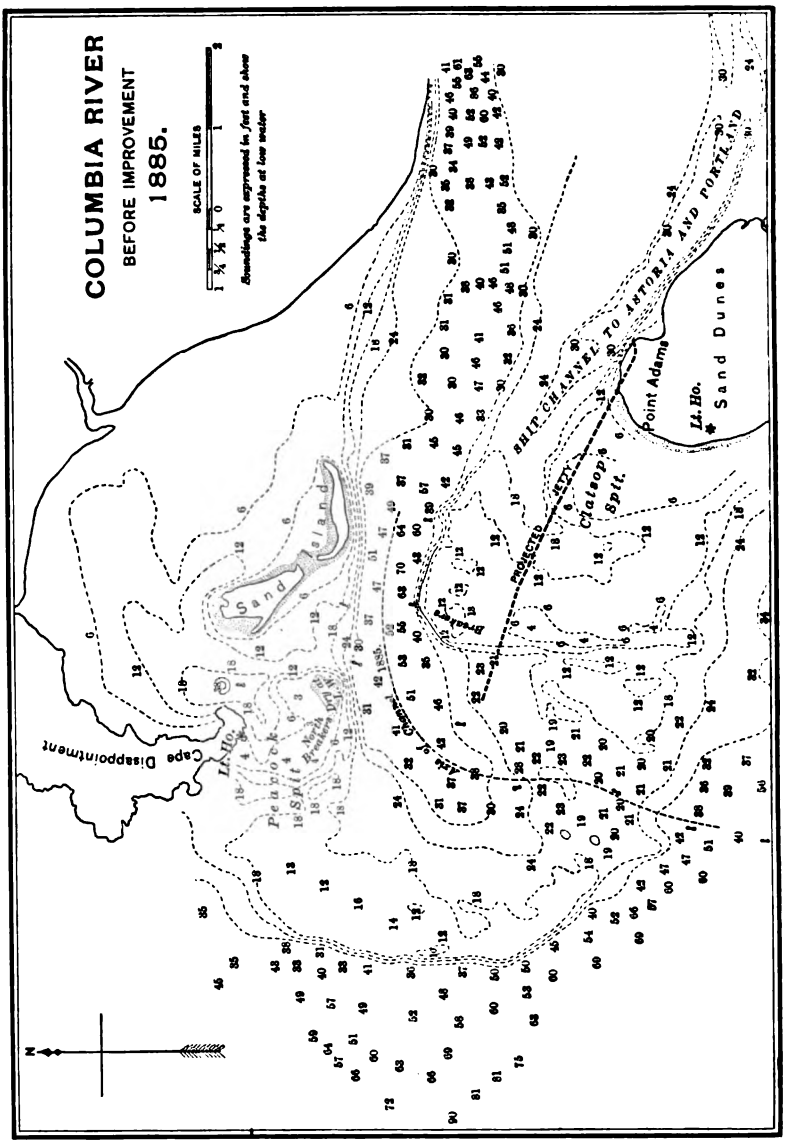

FIG. 4.

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Col. Mendell.

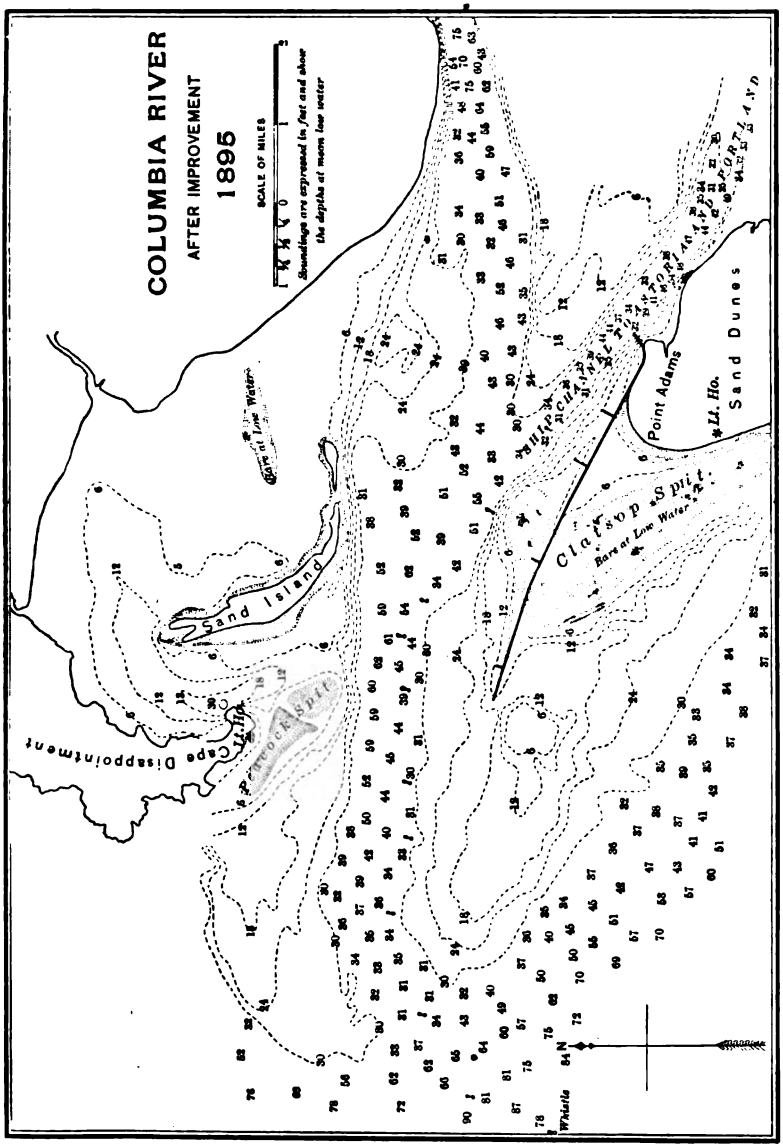

FIG. 5.

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Col. Mendell. sand yields to the slightest influence of wave or current, the Spit is variable in plan and elevation, and cannot of itself steadily fulfil the functions of a jetty. The sands on the north side, including Peacock Spit, have always been variable, but the rocky promontory of the Cape has given sufficient constancy to form a good northern bank equivalent to a jetty.

The stone line or jetty extends over Clatsop Spit seaward for nearly 5 miles. The specific purpose of this work is to give permanency to Clatsop Spit, holding it at as high a level as possible. The Spit also gives protection to the jetty on the weather side. But for the sand here accumulated the so-called jetty could hardly be held above low water, for it is necessarily built of country rock, of maxima weights of 7 or 8 tons, affording insufficient resistance to sea waves. At the western end the work has been flattened by waves to about low-water level, although built more than once to high water. The shore end for about 3 miles is held above high-tide level, gradually falling to mid-tide level. Generally described, the work is held as high as practicable under the exposure and with the available sizes of stone. While the line is fairly protected on the weather side by the shoals which it impounds, it needs special protection on the channel side against the ebbing current and against racing and enfilading waves from the sea. Being for the most part founded in shoal water and upon sand, the work is at the mercy of an undermining current or wave. It is therefore necessary to break up racing waves and to divert currents. As these sources of disturbance develop power to injure in deep water, it is necessary to keep deep water at a distance. Protection on the inside is provided by groins, which are also lines of stone work. The proper groin lengths depend upon, and are multiplies by a safe factor of, the difference in depth of the foundations and of the adjacent channel.

The jetty, to fulfil its functions best, must be tight, with no large interstices through which water and sand may circulate. It must therefore be made in great part of small stones, at least sufficient in quantity and varied sizes to reduce the dimensions of interstices to a minimum.

The dynamic principle of a work of this kind is that, to as great an extent as possible, the effluent tide shall be carried as a unit in a deep, direct channel, extended under control as far seaward as practicable. This principle may be applied with equal ease and safety to all volumes of tidal prism, howsoever great.

The stone lines are to be placed at a safe distance from the deep adjacent channel, and are to be guarded by groins of sufficient length. The channel which conveys the tidal efflux ought to be proportioned in width so as to maintain a low-water depth at least twice that expected to result at the bar. The deeper and more concentrated and

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more direct is this current, the longer after control is released will its stored energy remain and the better ought to be its expected action upon the bar. According to the view here taken, the spacing or distance between the stone lines plays no necessary part in securing the best results upon the bar. The great requirement is that the lines shall be protected, either by sand deposits or groins or by both. A slight convexity in alignment toward the channel side is also a safeguard against undermining, either by waves or by currents. Col. Mendell.

Whether the bar be due merely or mainly to littoral sand movement, or whether it be due to deposits made by waves, or, as obtains upon the Pacific Coast, to both causes, the best chance of counteracting these hostile influences seems to lie in a concentration of the force which makes for depth. An adequate tidal prism duly controlled for a conserving force, an abundance of sand for protection of the slight construction in country rock, and a fair slope in the sea floor, are main conditions which may be said to ensure success upon the Pacific Coast.

A single jetty cannot, in the general case, be said to fulfil the first condition. At most it does a partial service. It is good as far as it goes. It limits the azimuth of variation in channel alignment, but does not determine a single channel.

The conditions of the Pacific Coast are not favorable for success by dredging alone. The constant heave of the sea, the frequency of gales, the distance of the bar from shore, the continuous shifting of sand requiring steady excavation and the impossibility of ensuring it, seem to rule out dredging as a sole means of bar improvement, unless in exceptional instances.

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CORRESPONDENCE.

Capt. Abbot. F. V. ABBOT, M. Am. Soc. C. E.—The valuable paper on the Grays Harbor project brings out very forcibly what the writer believes to be the *sine qua non* of any successful improvement of a harbor entrance obstructed by a shifting sand bar, viz., interfere with existing conditions just enough to produce the desired local effect, but not enough to produce such extensive changes as to introduce new and unknown factors into the indeterminate equations between the forces tending to close up, and those tending to deepen and enlarge, the entrance. The bar itself is the resultant effect of all the forces acting on the movable sands of which it is composed. An attempt to oppose violently any of these forces will develop in the most convincing manner the vast potential stored in moving water. The attempt should be to concentrate and assist those forces which are favorable, and at the same time to oppose slightly those which tend to close the entrance.

Under the latter head the author does not mention one which has to receive most careful study on the South Atlantic Coast. This is a general drift of material along the coast. At a number of entrances on this coast, the drift is so enormous that a single jetty such as he proposes would be foredoomed to failure. On the Pacific Coast the material composing the shores appears to shift locally sometimes one way and sometimes in the opposite direction. Under such conditions there appears to be good reason to hope for a successful single jetty project.

The principal opposing forces are the flood tide and waves; the aiding force is the ebb tide, including the fresh water derived from the rivers discharging into the bay. The great difficulty is to modify the balance between these forces by a single jetty so that the ebb may overcome the combined energy of the flood and wave action to such limited extent as to produce the desired local deepening without altering the general regimen of the entrance.

The position and form assigned the south jetty being unquestionably the result of prolonged expert study of many facts and much data that could not be compressed into the narrow limits of a Society paper, it would be simply arrogant assumption to discuss it in detail, but it may be well to consider some points about it in a general way. The further the trace or ground plan is moved to the north, the greater the effect produced on the bar will be, and also the greater will be the danger of scour too near the structure. The form of jetty convex to the ebb is the exact reverse of that successfully used at Swinemunde. It is well adapted to securing immunity from scour, but to

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determine the degree of curvature demands engineering ability of the highest order. Too great a radius of curvature fails to protect against the formation of a channel too close to the structure; too small a radius enfeebles the effect at the very point, at the bar itself, where the maximum effect is needed. The southward shifting of the natural channel is the measure of the resultant force to be opposed, and the jetty must be strong enough in cross-section and trace to withstand this force successfully. The less the radius of curvature, the less the cross-section required to meet this attack, but the action of heavy waves may be increased if the curvature is too sharp. Capt. Abbot.

The further south the whole jetty is placed, the less is the danger of undermining, but also the effect is less on the bar. The cost is greatly affected by the depth in which the jetty is built, so that the configuration of the present bottom must be considered in addition to all the other factors that enter into the study. In a case of this kind mathematics must, as in many other cases of engineering study, be well subordinated to hard, practical common sense.

LEWIS M. HAUPT, M. Am. Soc. C. E.—The improvement of each harbor entrance is an independent problem, and the success or failure of the work must depend upon local agencies, yet there are certain general, fundamental conditions which must be complied with before success can be attained. The chief of these is the resultant direction of the littoral drift. On this point the paper seems to be somewhat ambiguous. It reads: Mr. Haupt.

"As the island travels north, the channel travels south. The entrance channel is at its best when leading out to the southwest. From this position it swings gradually around to the south, until it reaches a direction that is nearly due south. As this movement takes place the channel becomes deeper, but narrow and crooked, and immediately following this it breaks out to the west and commences to swing around to the southward again. * * * It is this movement or tendency which it is believed will make a single jetty interposed in its way effective in maintaining a permanent deep bar channel."

If the channel movements are as stated in this quotation, then the prevailing littoral drift must be in the same direction, and the proposed jetty will be on the wrong side to interpose an effective resistance to the forces forming the bar. Experience in jetty construction on the Gulf Coast leaves no doubt as to the injury and expense resulting from the improper conduct of such work. In one case, a bar was pushed out to sea at least 2 miles, adding 4 miles to the length of jetties to be built, merely because the littoral drift was not arrested on the near side of the channel. In short, the wrong jetty was built first.

At the mouth of the Columbia River the original design contemplated two jetties, as did the plan for Coos Bay, but the fortunate construction of the first jetty on the proper side of the entrance intercepted the littoral drift, conserved the energy of the ebb over the bar, and produced the required depth, rendering a second jetty unneces-

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Mr. Haupt. sary. Concerning the littoral current at Grays Harbor the author states: "There is a littoral current, the general resultant of which tends to the north. * * * Its intensity is as high as 2½ miles per hour either way," and its direction in the immediate vicinity of the land veers with the wind. While the draught current should be northerly from Cape Mendocino along this part of the coast, the southerly drift of bars enclosing the channels is a warning to look for some local eddy disturbance at this point, else the jetty will fail of its purpose and the experience on the Gulf will be repeated.

The writer's experience leads him to the conclusion that even the single jetty may be made much more effective by building it detached and concave to the ebb stream, so as to develop the centrifugal force of the currents locally over the crest of the bar. The laying of the foundations of the jetty at Aransas Pass, where there is a mean tide of only 14 ins., and the placing of a small portion of the superstructure produced phenomenal results. Here the usual order of procedure was reversed, and the work was commenced, as directed, on the outer slope of the bar to prevent any seaward movement and induce an immediate scour. The anticipated effects were more than realized. The report of the resident engineer, William Dunbar Jenkins, M. Am. Soc. C. E., showed that within a few months after the first mattress was laid, the outer and inner slopes were reduced and the crest lowered more than 3 ft., while the distance between the outer and inner contours was reduced nearly one-half.

What is needed in such improvements is a local treatment by a reaction breakwater extending across the crest of the bar on the side of the channel whence comes the littoral drift, so as to intercept it. This breakwater should have sufficient development shoreward to control enough of the ebb energy to cut a channel of the requisite depth without impeding the entrance of the tidal volume. Twin jetties, sufficiently close to act together, and running out from a harbor entrance where the tidal movement is as great as at Grays Harbor, would not only injure the inner channels, but would soon be destroyed unless a large sum was spent annually for their maintenance.

Although the jetty at the mouth of the Columbia River has accomplished more than was anticipated, it by no means follows that a similar jetty, similarly placed at Grays Harbor, will produce the same effect, because of the different conditions. Sufficient data are not given in the paper to permit of a definite solution of the problem, yet the writer believes that the author will be able, with the material accessible to him, to secure much better results at less than half the cost by employing a single jetty, placed correctly, rather than twin jetties. The writer's purpose in this communication is to call attention to the danger and disappointment which may result from placing the structure on the wrong side of the proposed channel.

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T. W. SYMONS, M. Am. Soc. C. E.—Naturally the first question that would arise in the mind of an engineer called upon to devise a plan for deepening the entrance to a harbor would be: "Can it be done by dredging?" Mr. Allardt's experience causes him to ask this question. To the engineers who have been engaged on the North Pacific Coast the method of improving a harbor entrance opening directly upon the ocean by dredging through the bar is now never seriously considered. No time is wasted upon it, as it early becomes evident to every student and observer that such a method is entirely impracticable. The sands move so freely and in such enormous volumes, under the influence of the frequent and terrific storms to which the region is subject, that no reliance can be placed upon any channel which is unable, with the existing forces, to maintain itself against all existing forces, and if necessary to renew itself. If it is conceived that at Grays Harbor a channel be dredged such as Mr. Allardt proposes, it is practically certain that the first severe storm would wipe it out of existence. Good natural channels across the bar have existed at many times, but they change in location and entirely disappear with remarkable rapidity. If these natural channels are not maintained, how could permanency be expected in an artificial channel 200 ft. or 300 ft. wide? Mr. Allardt's proposed method of cutting a channel is impracticable. No dredge with anchor moorings could live and work upon Grays Harbor bar, even in the mildest weather, for more than a few hours at a time, and for a very few days in a year. Maj. Symons.

The only possible way of dredging on the bar would be by the use of self-contained, sea-going, hydraulic dredges like those used on the New York bar, one of which is in use on the Columbia River for removing bars from the inner river. Even this could work only with great difficulty and risk, and for but a small part of the time.

If Mr. Allardt were familiar with local conditions, he would come to the conclusion of all other local engineers, which, as expressed by Col. Mendell, is that "the conditions of the Pacific Coast are not favorable for success by dredging alone," and this is particularly true along the northern part of the Pacific Coast of this country, where storms of great violence are of frequent occurrence and where the sea is almost always rough, the sand movements great, and the channels change in location with great rapidity and over wide areas.

The testimony of engineers is conclusive as to the existence of a well-defined littoral sand drift along the Atlantic and Gulf Coasts, which is permanent in direction, and which, of course, is a factor of prime importance in any scheme of improvement. On the Northwest Pacific Coast there is no such defined drift. The great sand movements are not due to a permanent littoral drift or current, as this is generally understood, but are due to local causes and to ever-changing winds and currents, which effect a distribution and redis-

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Maj. Symons. tribution of the sands existent about the harbor entrance and brought down by the tributary rivers.

The littoral current in front of Grays Harbor, as stated in the paper, runs both ways with varying intensity, and any work planned with the sole object in view of arresting littoral drift would be out of place there.

Prof. Haupt is in error in stating that the original plan for the improvement of the Columbia River contemplated two jetties. It only contemplated one, and estimates were made for but one, for the jetty which has been built. At Coos Bay two jetties were planned and estimated for. The first one built has been successful in developing and maintaining a bar channel fully equal to that which was hoped for from the two jetties. Even in the case of the work at Coos Bay the selection of the north jetty, as the first to be built, was not the mere fortuitous circumstance which Mr. Haupt infers it to have been. It was decided upon, after a careful consideration of all the prevailing physical phenomena, as the one which would do the most good, especially after it had been pushed out far enough to interpose an effective barrier to the constant northerly trend or movement of the entrance channel across the bar. Prof. Haupt is again in error in saying that "the jetty at the mouth of the Columbia River has accomplished more than was anticipated." The engineers who designed this work fully expected it to accomplish all that it has accomplished.

The action and function of these jetties is more complex than simply to arrest the littoral drift. This, in fact, is but a secondary function of either of the jetties; the primary function being to act as a directrix to the ebbing waters. Together with the natural tendencies of the channel to move in a certain direction, each develops a channel in a particular location and keeps it there.

In planning jetties for the improvement of a harbor on the Northwest Pacific Coast, and particularly in planning a single jetty, the trend and changes in the ebbing and flowing currents are factors of far more importance than the littoral drift, which Prof. Haupt seems to consider as of prime and sole importance.

The method of construction adopted at Aransas Pass would be inapplicable and impossible at Grays Harbor. The water is too rough to permit the construction of jetties from floating plant, as done along the Atlantic and Gulf coasts. One of the reasons for placing the jetty convex to the channel was to guard it against the very centrifugal forces of the currents which Prof. Haupt would develop. It is essential to the safety of a jetty built as this is planned, in a comparatively frail manner on a shifting, sandy foundation, to keep it as far away from the developed channel as possible. As the author endeavored to make clear, the volume of tidal flow at Grays Harbor is so great that but a small portion need be controlled in order to secure a channel

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over the bar of ample depth. Rules that would hold good in a place like Aransas Pass with a mean tide of 14 ins. might, and in some instances undoubtedly would, be disastrous, if applied at a place like Grays Harbor, with a mean range of tide of 8.4 ft., and a tidal discharge many times that at Aransas Pass. Maj. Symons.

Prof. Haupt further says: "Sufficient data are not given in the paper to permit of a definite solution of the problem." The answer to such a criticism is, of course, sufficient data are not given for a definite solution, and it would be impossible to give sufficient data to enable an engineer 3 000 miles away and unfamiliar with local conditions to solve the problem correctly. No engineer of high standing, familiar with such works, would dream of definitely solving the problem without visiting the locality and becoming familiar with conditions which cannot all be described or depicted.

Col. Mendell summarizes in a very clear manner the great principles underlying the methods of harbor improvement on the Pacific coast. The author does not disagree with anything he says, but goes little further than he does. In regard to a single jetty, he says: "A single jetty cannot, in the general case, be said to fulfil the first condition" (control of a tidal prism for a conserving force). "At most, it does a partial service. It is good as far as it goes. It limits the azimuth of variation in channel alignment, but does not determine a single channel." This is undoubtedly true, but there may be in addition to a single jetty natural forces acting with unvarying regularity, which in connection with a single jetty, determine, develop and maintain a suitable and sufficient channel. This is the case at Columbia River and at Coos Bay, and the author believes that it will be found to be the case at Grays Harbor. Why, then, should the invariable rule be adopted of providing double jetties, when nature has provided a substitute for one of them? The author's contention is that works should be planned so as to take advantage of the natural tendencies of the channels, so that, if possible, the desired results can be accomplished with one jetty. The way is always open to supplement this afterward with another jetty, if a more complete conservation of the tidal forces is necessary.

Captain Abbot, in his opening paragraph, states clearly the fundamental principle which must govern in any successful attempt to control such gigantic forces as those existing at the entrance to Grays Harbor, viz.: "Interfere with existing conditions just enough to produce the desired local effect, but not enough to produce such extensive changes as to introduce new and unknown factors into the indeterminate equations between the forces tending to close up and those tending to deepen and enlarge the entrance."

The points mentioned by Captain Abbot, as well as many others, were considered in the location of the jetty. Captain Abbot says "the

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Maj. Symons. cost is greatly affected by the depth in which the jetty is built, so that the configuration of the present bottom must be considered in addition to all other factors that enter into the study." While this is true to a limited degree, it is in reality not as important as it appears to be. It is found in practice that as soon as the jetty is pretty well out, that the sand scours away from its front, making a cross channel of approximately the same depth as the deepest water near it. The construction starts at the bottom of this cross channel. This scour can be regulated to a certain extent by the order of the work, but it is generally considered best to let it take place to a considerable degree in order to put the foundation of the jetty at a reasonably safe depth.

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SOME GENERAL NOTES ON OCEAN WAVES AND WAVE FORCE.*

By THEODORE COOPER, M. Am. Soc. C. E.

PRESENTED MAY 20TH, 1896.

WITH DISCUSSION.

A wave in a general sense has been defined as a state of disturbance propagated from one part of a medium to another. Through the oscillatory action of the particles of the medium, the energy due to a disturbance is transferred from one point to another. There is not, necessarily, a transfer of matter through the action of a wave, though in some cases the wave displaces, usually to a small amount only, the medium through which it passes. Currents, on the other hand, imply the passage of matter associated with energy.

Waves may be free or forced. A free wave is one which is no longer acted upon by the disturbing influence, either through the cessation of the disturbance or through the wave having outrun the sphere of disturbance. A forced wave is one upon which the disturbing influence continues to act so as to modify its propagation.

* A large portion of the following article has been collated from various sources at different times, and is not claimed by the author as original.

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Thus, while a gale is blowing the character of the water wave is continually being modified; beyond the immediate influence of a gale, or after it subsides, regularly oscillating waves or rollers are obtained, for the long ones not only outstrip the shorter ones, but are less speedily worn down by fluid friction.

The ocean tide is mainly a forced wave, depending on the continued action of the moon and sun. The tidal wave in an estuary or river is practically free, being almost independent of moon and sun, and depending mainly upon the configuration of the channel and the tidal disturbance at the mouth.

In the Thames, the mean range of the tide at Sheerness is about 18 ft.; at Deptford, about 17; at London Bridge, about 15; and from this point it diminishes gradually to the weir at Teddington where it is 2 ft.

At the entrance of Bristol Channel the whole rise at spring tides is about 18 ft.; at Swansea, about 30 ft., and at Chepston, about 50 ft.

At the entrance of the Bay of Fundy, the currents are very rapid, but the range is only about 8 ft.; at the head of the bay, where it branches into two shoal arms, the range of the tide sometimes amounts to 60 to 70 ft.

At Cherbourg, it is only 15 ft., but at St. Malo it rises nearly to 40 ft.

The most usual division of free waves is into long waves, oscillatory waves and ripples. The first two classes run by gravity and the third mainly by surface tension. Long waves, or waves in water that is very shallow, compared with the length of the wave, are due to disturbances which extend to the full depth of the water. Oscillatory waves are due to disturbances confined to the upper layers of the water, which die away with great rapidity in successive layers below.

Mr. Scott Russell called the long wave a "wave of translation," in contradistinction to a "wave of oscillation." Under waves of translation he included the tidal wave, bores,* ordinary ground swell and waves

* Airy says: "We believe the following description of the cause and appearance of the bore will be found correct. It is necessary for its formation that there be a very large tide rising with great rapidity (thus at Newnham, where the water rises 18 ft. in an hour and a half, the bore is considerable). It is necessary also that the channel of the river be bordered with a great extent of flat sands near to the level of low water. These circumstances hold in the Severn, the Seine, the Amazon, the bays at the head of the Bay of Fundy (Chignecto Bay and Bay of Mines), and other places where the bore is remarkable. When the rise of the tide begins, the surface of the water is disturbed in mid-channel, but the water is not broken, it is merely a common wave. But as this rapid rise elevates the surface suddenly above the level of the flat sands the water immediately rushes over them with great velocity and a broken front, making a great noise. This is the whole of the bore; it is, however, a majestic phenomenon."

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of oscillation when they reached much shallow water. Under waves of oscillation he included all other storm waves.

This classification has been adopted by many subsequent writers. This division, however, has no value as a classification, and is very apt to induce erroneous ideas on the action and power of waves. It is recognized now that all storm waves are waves of translation to a more or less extent, according to the strength of the wind, the time it has been blowing, and other circumstances.

Mr. Scott Russell, in describing his wave of translation, says:

"As the wave advances the particles of water of which it is composed, instead of continuing to travel forward themselves, force other particles upwards into the wave and themselves subside and take the place of these latter particles. These again behave themselves similarly, depositing themselves in the place of further particles; and in this manner the wave proceeds constant in shape and velocity but continually recomposed of new particles of water which are successively lifted up, carried forward and let down again into new positions.

"A solitary wave of elevation obviously carries across any fixed transverse section a quantity of water equal to that which lies above the undisturbed level. If the wave had been one of depression, the translation would have been in the opposite direction to that of the wave motion. Hence, when the wave consists of an elevation, followed by a depression of equal volume, it leaves the water as it found it."

The theory of motion of such a wave is based on the hypothesis that all the particles in a transverse section have at the same instant the same horizontal speed, the vertical motion of the water depending on the change of horizontal speed from section to section. The speed of propagation of such a wave of translation is given by Mr. Scott Russell as that which would be acquired by a body falling through half the depth of the water,

$$V = \sqrt{g\,h}$$

h being the depth of water measured from the wave crest.

Rankine states that long waves or waves in water that is very shallow compared with the length of the wave have a velocity nearly independent of the wave length and nearly equal to that acquired by a body falling through a height equal to half the depth of the water added to three-fourths of the height of the wave.

$$V = \sqrt{g\,(h^1 + \tfrac{3}{4}\,D)},$$

h^1 being depth of still water and D the wave height.

These two formulas are identical, for the wave crest is about three-fourths of the depth of the wave above still water, so $h = h^1 + \frac{3}{4}\,D$.

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The tidal wave travels at different speeds in different seas and oceans, dependent upon the depths of the sea. In the North Sea it travels 45 miles per hour. The Atlantic tidal wave (long wave) passes over 90° of latitude in 12 hours, or at a rate of speed of 520 miles per hour. The great earthquake wave of 1868 was estimated to have traveled in the Pacific Ocean at a rate of 300 to 400 miles per hour. The great wave formed in the Straits of Sunda by the volcanic eruption of Krakatoa was felt upon the coast of France, and its speed estimated at over 300 miles per hour.

The nearest example to a typical oscillatory wave is found in what is called the swell, or regular rolling waves, which continue to run in deep water after a storm. In a pure oscillatory wave, the motion of each particle is in nearly a circular orbit; the particles at the surface of the water describe the longest orbits; the extent of the motion, both horizontal and vertical, diminishes in a geometrical progression as the depths increase in arithmetical progression and at great depths the motion is insensible.

Fig. 1 is Weber's diagram of an oscillatory wave, Fig. 2 showing the same filaments in still water, *O O* being the level of the surface in still water. The particles of water are rotating to the right in circles whose radii vary from r equal R for the surface particles, to r^1 equals $r\,e^{-\frac{k}{r}}$ for all particles at a lower depth, k being the depth of the filament below still water, e the base of the Naperian logarithms, r and R as given later, and $-\frac{k}{r}$ being an exponent to e.

The trachoidal lines *O*, *a*, *b*, *c*, etc., in Fig. 1, represent the form taken by the filaments *O*, *a*, *b*, *c*, etc., in Fig. 2 during the passage of a wave. The left-hand portion of Fig. 1 shows the distortion of the filaments, both horizontally and vertically, this being greatest at the surface and diminishing rapidly as the depth increases. The wave is traveling to the right, or in the same direction as the particle at the top of its orbit. The energy imparted to the particles is expended in lifting the particle the distance of its center of rotation above its position at still water, and in giving it rotation about the center. It can be shown that one-half of the energy of disturbance goes to lift the particle to its mean height and one-half to impart rotation.

Figs. 3 and 4 have been prepared to show the form of wave generated when the particles travel in elliptical orbits. Both the waves

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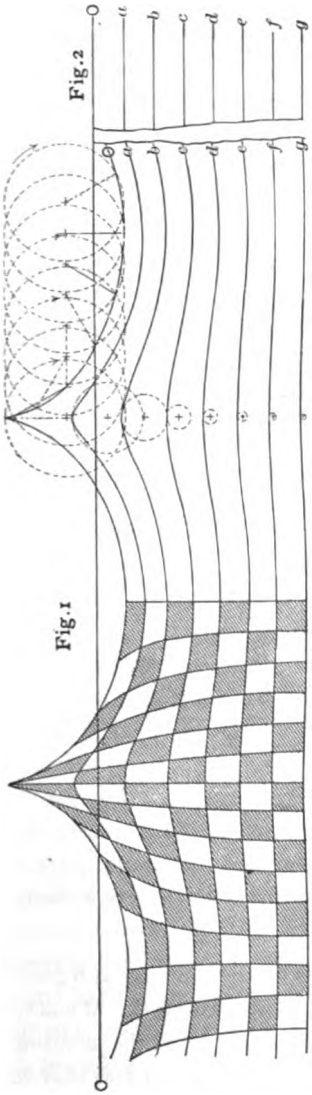

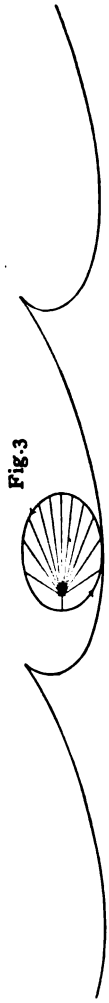

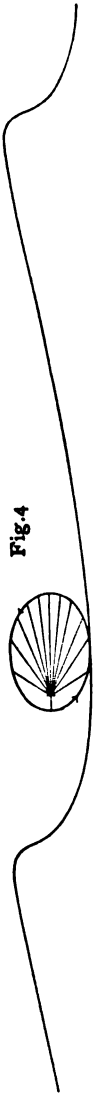

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shown are generated by the same ellipse, but the wave length in Fig. 4 is made double that of Fig. 3.

The following formulas (condensed by Prof. C. S. Lyman) are those giving the relations between the elements of such an oscillatory wave :

l = length of wave from crest to crest.
D = depth from trough to crest.
V = velocity of propagation of the wave.
t = period or interval of time between consecutive wave crests.
r = radius of orbit of a particle at the surface.
v = velocity of a particle on its orbit or at the crest of a wave.
R = radius of circle whose circumference is a wave length, also the length of a pendulum keeping time with the wave.
a = elevation of crest above still water.
b = depression of trough below still water.
g = acceleration of gravity.

$$l = t\,V = 2\,\pi\,R = \frac{g\,t^2}{2\,\pi} = \frac{2\,\pi\,V^2}{g}.$$

$$D = 2r.$$

$$t = \sqrt{\frac{2\,\pi\,l}{g}} = 2\,\pi\sqrt{\frac{R}{g}} = \frac{l}{V}.$$

$$V = \frac{l}{t} = \frac{g\,t}{2\,\pi} = \sqrt{\frac{g\,l}{2\,\pi}}^{*} = \sqrt{g\,R} = \frac{g\,r}{v}.$$

$$v = \frac{2\,\pi\,r}{t} = r\sqrt{\frac{g}{R}} = r\sqrt{\frac{2\,\pi\,g}{l}} = \frac{V\,r}{R} = \frac{2\,\pi\,r\,V}{l} = \frac{g\,r}{V} = \frac{t\,g\,r}{l}.$$

$$a = r + \frac{\pi\,r^2}{l} = r + \frac{r^2}{2R}\,;\; b = r - \frac{\pi\,r^2}{l} = r - \frac{r^2}{2\,R}.$$

Sine of angle of steepest slope is sin. $\theta = \frac{2\,\pi\,r}{l} = \frac{r}{R}$.

$R : r = g : \frac{4\,\pi^2\,r}{t^2}$, or as gravity is to the centrifugal force of a surface particle.

The maximum height of a wave depends on the centrifugal force of the particles, and thus on the external forces generating it. When the centrifugal force becomes equal to gravity, or $r = R$, the resultant force for a particle at the highest point of its orbit or at the crest of the

* $V = \sqrt{\frac{g\,l}{2\,\pi}}$ is the velocity due to a heavy body falling $\frac{l}{4\,\pi}$ or as it is sometimes worded, approximately one-thirteenth of the wave length.

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wave must be zero. Beyond this point the particle flies from its orbit and the crest breaks into foam.

The greatest elevation of the crests above the level of still water is also when $r = R$ and equals three-fourths of the depth of the wave, and the centers of the orbits of the surface particles will be on a line one-fourth of the depth of the wave above the level of still water.

Capt. Gaillard, from 45 observations on waves 2½ to 6 ft. deep just before breaking, found that the elevation above still water varied from 0.67 to 0.89, with a mean of 0.76; with a flat slope of the bottom and opposing wind it was increased, while with a steep slope and favorable wind it was decreased.

Stevenson says waves at Wick's Bay were two-thirds above and one-third below mean level.

Rankine gives elevation of crest above still water $= \frac{D}{2} + 0.7854 \frac{D^2}{l}$

$$\text{depression of trough below still water} = \frac{D}{2} - 0.7854 \frac{D^2}{l}$$

The height of the wave is, within certain limits, independent of its length.

The longer waves travel the faster.

The normal oscillatory wave to which the preceding theoretical equations apply is the wave on deep water or water more than a wave length in depth. In shallower water the orbits are no longer circles, but approximately ellipses of less height than length according to the degree of shallowness.

When a series of waves advances into water gradually becoming shallower, their periods remain unchanged, but their speed, and consequently their length, diminishes and their slopes become steeper. The orbits become distorted in such a manner that the front of the wave becomes steeper than the back, the crest, as it were, advancing faster than the troughs. At length the front of the wave curls over beyond the vertical, its crest falls forward and the wave breaks.

Purely oscillatory waves rarely, if ever, occur in the ocean. They are also more or less waves of translation. The forward horizontal motion of each particle under the crest is not quite compensated for by its backward movement under the trough, so that the wave sets down each particle of water a little in advance of where it picked that

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particle up, causing what is known to sailors as the "heave of the sea." This effect, however, like the whole disturbance, is greatest at the surface layer and diminishes rapidly for each lower layer.

Long waves are analogous to oscillatory waves with the modification that, owing to the enormous length of the waves as compared with the depth of the water, the extent of horizontal motion of the particles is nearly equal at all depths and the extent of the vertical motion in any layer is nearly in the simple proportion of its height above the bottom. The orbit of such particles is a very long and flat ellipse. The deeper a particle is situated, the flatter becomes the orbit, a particle in contact with the bottom moving backward and forward in a horizontal line.

Comparing the equations for the speed of propagation of a long wave, $V = \sqrt{g\,h}$, with that for an oscillatory wave, $V = \sqrt{\frac{g\,l}{2\,\pi}} = \sqrt{g\,R}$, it will be seen that they are very similar in form.

Rankine gives as a general equation for the velocity of waves of all figures and for every kind of disturbance, provided only that the upper surface of the water is free:

$$V = \sqrt{g\,k}$$

k being the virtual depth of uniform disturbance, or, more briefly, the virtual depth, or a depth which, with a uniform velocity equal to the horizontal surface velocity of a particle, would give the same volume of horizontal displacement as occurs at a vertical plane through the crest of a wave with the varying velocities of any wave.

For an oscillatory wave, $k = R$, and for a long wave, $k = h$.

When oscillatory waves pass from deep to shallower water, imparting their energy to a less and less mass of water, and flattening their orbits, they take on more and more the characteristics of long waves. Mr. Scott Russell says that if waves be propagated in a channel whose depth diminishes uniformly, the waves will break when their height becomes equal to the depth of the water. He has never seen a wave of 10 ft. in 10 ft. of water, nor so much as 20 ft. in 20 ft. of water, nor 30 ft. high in 5 fathoms of water, but has seen them approach near these limits.

M. Bazin has shown that a wave breaks when its height exceeds two-thirds of the total depth. The most trustworthy measurements of waves in the mid-ocean show 44 to 48 ft. to be a remarkable height.

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Waves having a height greater than 30 ft. are not commonly encountered. Scoresby measured waves 43 ft. high, 559 ft. long, traveling at a velocity of 32½ miles per hour, at 16-second intervals.

Dr. Schott (1891–92) reports the greatest storm wave observed by him as 36 ft. above sea level. Lieut. Paris records 37 feet above sea level. Dr. Schott says the heaviest storm waves he met moved about 58 ft. per second, and attained a length of about 650 ft. He confirms Lieut. Paris' determinations that the proportion between wave height and wave length in a high sea is as 1 to 18, and in a moderate sea as 1 to 33. Capt. Gaillard, from 58 observations at St. Augustine on waves varying from 4 ins. to 6 ft. in height, found that the depth at which waves broke varied from 0.72 to 2.0 times the wave height, but that for the greater number, it equaled the wave height. A strong, favorable wind made it equal 1.25, while an equal wind in the contrary direction caused it to fall to 0.72; when there was no wind and the bottom was uniform and sloping only 1 in 100, the depth was nearly always 1, but when the slope was about 1 in 12, it became 2. Officers of the United States Navy have recorded waves measuring ¼ mile from crest to crest and a period of 23 seconds.

Waves having lengths of 500 to 600 ft. and periods of 10 to 11 seconds, are the ordinary storm waves of the Northern Atlantic.

Mr. Deverell observed in the equatorial calms a swell, which was never absent and measured 4 ft. high, 500 ft. long, and had periods of 10 seconds.

The study of waves and wave action is one of great importance to engineers.

The strength, modeling and stability of vessels, the best speed of boats in channels of limited cross-section, the wash of banks in canals and impounding reservoirs, the protection of sea coasts, the improvement and maintenance of channels and harbors, the design and maintenance of artificial structures which are liable to wave action, whether they be inland and subject to the sudden onslaught of a wave produced by the rupture of some restraining medium, or located on the coast and subject to the attack of sea waves—all require a knowledge of water waves and their action.

This paper will be confined to the consideration of the action of waves or the force of the sea upon artificial structures.

Smeaton classed the action of the sea as among the powers of nature

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that are not subject to calculation. Since his time, experience with sea structures and knowledge of the laws governing the force of waves have been greatly extended. The valuable collection of facts and experiments by Mr. Thomas Stevenson and others upon the power exerted by sea waves, and the study of the laws governing the action and forms of waves by Messrs. Scott Russell, Rankine, Stokes and many other able investigators, go far to enable a reasonable estimate to be made of the forces to be encountered by structures subject to wave action.

Mr. Scott Russell says:

"In sea works there were practically two classes of waves to deal with. In deep water there were not only the oscillatory surface waves to be encountered, but also those which he had termed 'waves of translation,' forming what were called 'rollers,' or, when on a smaller scale, 'ground swell.' These were a much more troublesome class of waves. It was mainly with them that the engineer had to deal in places open to the Atlantic.

"To reflect or send back the roller was the most effectual plan. For this purpose nothing more was necessary than a deep perpendicular face of perfect masonry, and so long as it stood firm it was faultless, for the reflection really converted the whole effect of the roller on itself into a simple pressure of water. When such a wave was reflected on a perpendicular wall, it merely provided a hydraulic pressure equal to that due to little more than double its own height."

Rankine says:

"When waves roll against a vertical wall they are reflected, and the particles of water for a certain distance in front of the wall have motions compounded of those due to the direct and reflected waves. The particles in contact with the wall move up and down through a height equal to double the original height of the wave."

This is in accordance with well-known mechanical laws. The work stored in a wave must necessarily be measured by the weight of water in the wave, multiplied by the mean distance to which this mass of water has been lifted. The height to which each particle is lifted is determined by the centrifugal force due to the velocity of its rotation. The energy of each particle will be measured by its weight and its velocity of rotation, or, in other words, by its weight and the height to which it will be lifted by the centrifugal force.

To destroy such a wave, each particle must be opposed by a force equal and opposite to that which it has. To reflect or throw it back an additional opposing force of the same amount must be added to it.

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A column of water, then, double the wave height would measure the maximum force necessary to reflect the particles in the maximum section through the wave crest. Let Fig. 5 represent a long or solitary wave of elevation (a wave of translation).

$O\ O$ = level of undisturbed water.

a = elevation of wave crest above sea level = $\frac{1}{2} D$.

h = depth of water.

V = velocity of propagation of the wave.

v = uniform horizontal velocity of water in the section $A\ B$.

Then $V : v = h : D$ or $v = \frac{V D}{h}$, but $V = \sqrt{g\ h}$,

therefore $v = D \sqrt{\frac{g}{h}}$.

The vis viva of all the particles in the section $A\ B$ will be:

$\frac{1}{2} M v^2 = \frac{M g D^2}{2 h}$ where M is the mass of the particles in the section $A\ B$.

$M g = W = \gamma h$ = weight of particles in section $A\ B$.

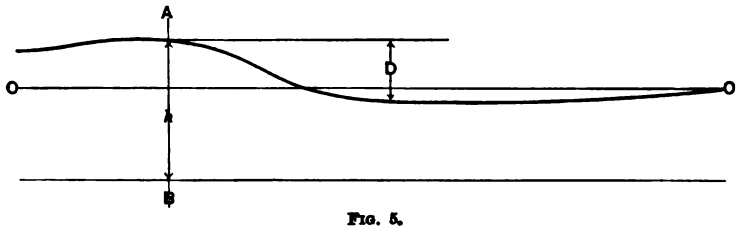

FIG. 5.

Substituting in above, vis viva = $\frac{1}{2} \gamma D^2$; but γD is the weight of particles for a depth equal to the wave height. Hence, the vis viva of such a section, $A\ B$, of a long wave, where the horizontal velocity is uniform for the full depth of the water, is measured by the product of the weight of water γD of the wave section and its mean height $\frac{1}{2} D$, which is identical with that of a column of water of this height. To destroy suddenly the energy of such a wave, it is therefore necessary to present an obstruction capable of resisting a head of water double that of the wave height.

From the practical side, there is a mass of facts and experiments upon the force exerted by the sea which is very impressive and tends to create doubt in regard to the correctness of theoretical deductions. While theories should be tested and judged by a comparison of their

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deductions with observed facts and the results of experiments, these latter should be carefully examined and received with much caution when they appear to contradict well-proved mechanical laws. Before questioning the correctness of the theoretical deductions, it would be well to examine the observations and experiments, to see if there are not other elements to be considered, which would explain the phenomena in accordance with true mechanical principles.

The text-books present as evidence of the enormous power of the waves the experiments of Mr. Thomas Stevenson by means of a dynamometer, the transportation of large masses of rock or masonry, the quarrying of rock from its bed, the great elevations to which spray or even solid water is thrown by the breaking of waves against cliffs or artificial structures, etc. An examination of the recorded facts should be made to see if proper and full consideration has been given to the various elements effecting the interpretation of such phenomena.

Stevenson by means of his marine dynamometer obtained at Skerrymore Rocks in the Atlantic Ocean the following average and maximum results in pounds pressure per square foot of surface exposed.

Average of summer months, 1843 and 1844....	611 lbs.
" winter " "	2 086 "
Maximum, March 29th, 1845.....	6 083 "

In the German Ocean he obtained the following pressures per square foot:

Maximum at Bell Rock.....	3 013 lbs.
" Dunbar.....	7 840 "

The dynamometers had disks from 3 to 9 ins. in diameter, but generally 6 ins.; the strength of the springs varied from 10 to 50 lbs. for every $\frac{1}{4}$ in. of elongation.

The pressures obtained by Stevenson are constantly used and accepted as evidence of the force of the waves, and even so careful and reliable an author as Rankine gives them as such, neglecting or overlooking the very important qualifications made by Mr. Stevenson in regard to them.

It is well to note here that Rankine apparently is unwilling to accept these high pressures on any other theory than that the waves did rise high enough to produce such high hydrostatic pressures, for

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he says, "the concentration of energy upon small masses of water produces waves of heights greatly exceeding those which occur in water of uniform depth, as the following examples show"; he then tabulates opposite the above pressures the feet of water which would produce them.

The following quotations show Stevenson's own opinion as to the applicability of these pressures as the measure of the wave forces. Referring to his observations, he says:

"It will be sufficient here to state generally the results obtained, only premising that the values refer to areas of limited extent, and are applicable therefore only to the *piece-meal* destruction of masonry, and must not be held as applicable to large surfaces of masonry.

"The experiments at Dunbar and Buckie prove that the sea may exert a force so great as 3½ tons *over the limited extent of surface presented by the disks*, and that the force varies much with the level at which the instruments are set."

The emphasis in all quotations is that of the author quoted.

Clearly, therefore, Stevenson does not accept these dynamometer pressures as the measure of the force of a wave, but solely as local pressures which can be produced by the waves. He gives the following records of great heights to which spray and solid water have been thrown by the action of the sea:

"On February 15th, 1853, during a gale a large body of water was thrown upon the lantern of Nosshead Lighthouse at a height of 175 ft. above the sea.

"At Unst, the most northerly of the Zetland Islands, a door was broken open at a height of 195 ft. above the sea.

"In November, 1827, during a ground swell *without* wind, the water rose to the gilded ball of the Bell Rock lantern 106 ft. It therefore follows that there is a force in action at the foot of the Bell Rock tower competent during ground swells when *there is no wind* to project a column of water to the height of 106 ft., which, according to the laws of hydrodynamics, is due to a pressure of very nearly 3 tons per square foot; whereas, the greatest force that happened to have been actually measured was only 1½ tons.

"It may be added that the spray on an average of 17 observations taken roughly was found to rise on a hollow curved wall about seven times higher than the waves which projected it; and on a vertical wall, taking a mean of 23 observations, it rose 6.6 times."

Now, if it had happened that the water which was thrown to those great heights had been thrown against the surfaces of the disks, the dyna-

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mometers, as Mr. Stevenson suggests, would have recorded the much higher pressures due to the height of those columns of water. Such pressures, however, would not measure the force of the waves, for only a small portion of the water in the wave was projected to those heights.

That the energy of a wave, upon meeting an obstacle like a wall or tower, would be expended in giving a greater velocity to those particles near the free surface and in projecting them in the direction of the least resistance, is not mysterious. The action of a hydraulic ram in lifting water higher than the head of the operating current is precisely similar. To determine the energy of such a wave, it is necessary to know not only the number of particles that are raised and their velocities, but also the greatest number of particles that are finally stopped.

That such a concentration of energy into a small portion of the water can cause very high local pressures, which should be considered in the detail of the work is unquestioned, but these pressures cannot be taken as the measure of the effect of a wave upon the stability of a large mass of masonry. They would bear about the same relation to the action of a wave, considered as a whole, as the local action of an engine wheel in crushing or indenting a rail would to the action of an engine or train load upon a bridge structure.

In addition to the records of the dynamometer and of the heights to which water has been thrown, other evidence as to the immense power of the sea has been presented, consisting of the movement of heavy blocks of stone, the breaking out or quarrying of rock from its bed, the breaking of timber booms, etc. The conclusions drawn from many of these instances, where sufficient data have been furnished as a basis of judgment, are, to the author's mind, entirely fallacious.

The cautious investigator would put little reliance on a calculation of the force of a wave from the breaking of wet timbers by repeated blows. The breaking out from their beds of blocks of stone or rock in place by the impact of a wave upon water already contained in the seams is a well-recognized process which in no manner conveys any knowledge of the force of the wave. The movement of masses of rock 6 to 13 tons in weight from their original beds 70 to 75 ft. above the sea level, as noted by Mr. Stevenson at Whalsey Skerries, Shetland, is very impressive. The action, however, is not mysterious, when the immense waves which occur at such places are considered. At Round Island, Scilly Group, large masses of water have been driven over the

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summit of this rocky island, which is 130 ft. in height, and small 2½-ounce stones carried to the top of a dwelling house 143 ft. above high water. At Bishop Rock Lighthouse, about 5 miles from the island, waves 50 ft. above high water have been noted. These waves would probably be about 66⅔ ft. from crest to trough. That waves of this size breaking upon a sloping rock or against a lighthouse should produce the destructive effects recorded is not astonishing, but that writers on wave force should assume these extraordinary results as some inherent force due to waves regardless of their height is surprising.

The author, while accepting all these evidences of the force of the sea at various localities, desires to show that they are due to the size of the waves at each locality, and, therefore, when the actual maximum waves are known, it is possible to estimate the resistance which must be given to every structure to meet the force of such waves.

Mr. Stevenson, after enumerating many wonderful effects of sea waves, all within his own personal observations, says:

"We should never expect that examples of the development of the *greatest* force would be found to be against masonry of those artificial works which form our ports and harbors. Accordingly, the examples of the most violent wave action which have been heretofore mentioned were cases of the dislocation or movement of dislocated natural rocks.

"This, however, no longer holds true. The most startling example now on record is that of an artificial work."

He then describes the Wicks Breakwater and the struggle to maintain it against the sea. For a full report of this work see Mr. Stevenson's reports. It will be sufficient for the present purpose to make the following extracts:

"In December, 1872, an enormous mass of concrete, at the end of the work, weighing not less than 1 350 tons, succumbed to the force of the sea, and the resident engineer actually saw it from adjacent cliffs being gradually 'slewed' round by successive strokes, until it was finally removed and deposited inside of the pier, having sustained no damage but a slight fracture of the edges. It was composed of three courses of large blocks of 80 to 100 tons (see Fig. 6), which were deposited as a foundation on the rubble. Above this foundation there were three courses of large stones carefully set in cement, and the whole was surmounted by a large monolith of cement rubble measuring about 26 ft. by 45 ft. by 11 ft. in thickness, and at 16 ft. to the ton, weighing upwards of 800 tons. This block was built *in situ*. As a

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further precaution, iron rods 3½ ins. in diameter were fixed in the uppermost of the foundation courses of cement rubble. These rods were carried through the courses of stonework by holes cut in the stone, and were finally embedded in the monolithic mass which formed the upper portion of the pier."

The shaded portion of Fig. 6 shows the part removed by the waves. This mass weighed no less than 1 350 tons, and presented an area of about 496 sq. ft. to the sea. The lower or foundation course of 80-ton blocks, *b b*, retained their positions unmoved. The second course of cement blocks, *a a*, on which the 1 350 tons rested, were swept off after being relieved of the superincumbent weight, and some of them were found entire near the end of the breakwater.

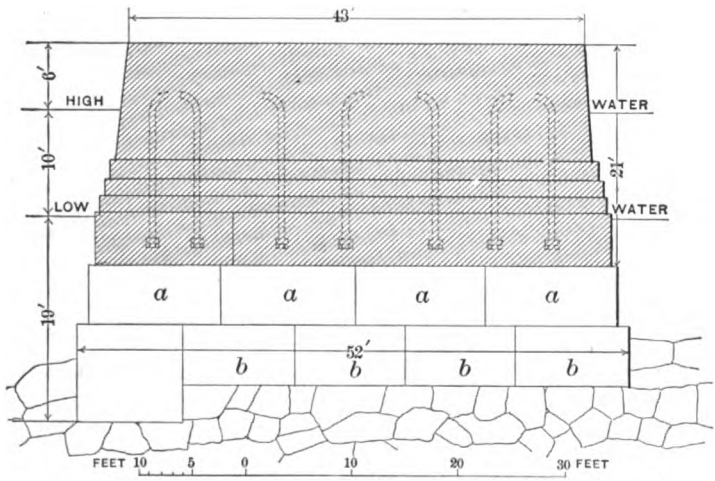

FIG. 6.

The waves by which the works had on various occasions been assailed, and which had carried away the outer portion of the work, were estimated by the resident engineer at 42 ft. from crest to hollow. They passed over the top of the parapet in masses of solid water estimated to be from 25 to 30 ft. deep, and, as ascertained by photographic views, the clouds of spray were projected to a height of not less than 150 ft. If it is assumed, as seems perfectly justifiable, that the horizontal bed joints along the top of blocks *a a*, which were about 5 ft. below low water, were not water-tight joints, the moving of this great mass by the waves of the above description does not appear so extraordinary. The buoyancy of the water would reduce the effective

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weight of this block when submerged in the wave to about 730 tons. With a frictional resistance on its bed as high as 75%, a pressure per square foot of surface of about 2 500 lbs. would be sufficient to move the block. The maximum pressure exerted by a wave 42 ft. high would be theoretically about 5 400 lbs. per square foot. It does not appear necessary, therefore, to assume any more mysterious force possessed by such a wave than would be expected by theoretical considerations, to explain its action in this case. In fact, as was afterwards the case, it would not be surprising to find even a larger mass of masonry moved by it.

In 1877 another concrete mass containing 1 500 cu. yds. of cement rubble and weighing about 2 600 tons, substituted for the one just described, was in like manner carried away.

In considering this case a friction of 75% has been assumed for this mass upon its bed. It seems very probable to the author's mind, however, that the impact of the waves upon the water in the bottom seams would greatly assist the other displacing forces.

In Volume CVIII of the *Proceedings* of the Institution of Civil Engineers, there is a very interesting description of the lighthouses built at Bishop's Rock, Scilly Islands. The first, a skeleton iron lighthouse was entirely swept away February 5th, 1850. This was replaced by a masonry lighthouse in 1858. In a brief time the great effects of the sea were noted. During a severe storm a fog-bell weighing 5 cwt. was torn from its bracket on the lantern gallery, 100 ft. above high water.

"When a hurricane of this type was raging, the tower vibrated violently; articles were shaken from the shelves in the rooms and the prisms of the dioptic apparatus fractured. Later on it was discovered that some of the external blocks of granite situated a few feet above high water had been split by the excessive strain imposed on the building.

"The tower was then strengthened from top to bottom by bolting heavy iron ties to the internal surface of the walls and connecting them through the floors. Nevertheless, after a violent storm in 1881, there was again evidence that the heavy strains of the sea had caused further damage. In certain places pieces of granite weighing ½ cwt. were split from the face of the exterior blocks a few feet above high-water level."

It was therefore resolved to strengthen and improve the tower. As it was apparently on the verge of destruction by the storms to which it

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had been subjected, it will be interesting to apply the preceding theoretical views to it. Sir James N. Douglas says:

"After the completion of the second work, they had more detailed information through the light-keepers as to the exact movement of those heavy seas. The primary wave flowed freely over the rock at a height of 50 ft. above its summit. There was very little break in the wave except on the tower itself; and there a rather lighter wave arose and reached the cavetto of the lantern gallery (100 ft. above high water), where a third and still lighter wave was produced of heavy spray, which struck the lantern and rising far above it, was driven a long way to leeward, sometimes nearly ¼ mile during a heavy storm. He had visited many parts of the world, east and west, but had never seen heavier seas than he had witnessed at Bishop's Rock, where everything appeared favorable to their development."

This tower was 36 ft. in diameter at the high-water line, and for 50 ft. above high water had an exposed wave surface of 1 320 sq. ft., and above the 50 ft. an exposed wind surface of about 1 275 sq. ft. (dimensions scaled from drawings). The cubic contents of masonry was about 38 000 cu. ft.; the weight of the tower and contents was estimated at 6 500 000 lbs., equal to an insistent pressure of about 6 400 lbs. per square foot on the base.

A wave which rose 50 ft. above the rock, which was about high water, would measure at least 66⅔ ft. from crest to trough. Such a wave reflected from a flat vertical surface would exert a pressure due to the mean of a water column 133⅓ ft. high, but 16⅔ ft. of this would be below the base of tower, so only 116⅔ ft. will be taken.

For a circular tower, there are theoretical grounds for believing that the pressure will be only one-third of that upon a continuous wall. Then there will be a mean pressure of $116\frac{2}{3} \times 64.3 \div 3 \times 2 = 1\,249$ lbs. per square foot of exposed surface; $1\,249 \times 1\,320$ sq. ft. $= 1\,648\,680$ lbs., which gives an overturning movement of about $1\,648\,680 \times 17 = 28\,027\,560$ foot-pounds.

Forty pounds of wind pressure acting on the upper surface of the tower would increase this moment 2 000 000 foot-pounds, making a total moment of 30 027 560 foot-pounds. The moment of resistance of the base would be $\frac{I}{z} = \frac{\pi \times 18^3}{4} = 4\,580.$

$$\frac{30\,027\,560}{4\,580} = \pm 6\,556 \text{ lbs.}$$

causing a reduction of the pressure on the base at the windward side

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from 6 400 lbs. to a tension of 156 lbs., and an increase on the leeward side from 6 400 to 12 956 lbs.

No masonry structure could be expected to stand such conditions of strain for a long time.

If, then, it is possible to explain these extreme cases of sea force upon well-recognized mechanical principles, it is safe to accept these same principles as sufficient for estimating the forces which sea structures will have to encounter. In estimating the forces to be resisted by a sea structure, the greatest waves which will possibly impinge against the structure when built must be determined. These may be entirely different from the off-shore waves. It will be necessary to study all the conditions of the location to determine the probable maximum waves which may have to be encountered, if the maximum waves at that point are not already known.

The off-shore waves running into shallower water or converging into narrower limits by the form of the coast or bottom will impart their energy to smaller masses of water, producing waves exceeding those originating them. Or on the contrary, the character of the bottom or foreshore and depths of water may be such as to cause these off-shore waves to break and lose their energy before reaching the structure.

It may be well to quote further from Mr. Thomas Stevenson:

"The determination of the stability of a practically monolithic mass, such as the Eddystone or Bell Rock lighthouses, though difficult enough, is, however, of a simpler nature than that of the disconnected materials of a harbor work. In the masonry of lighthouses the sea is excluded from the joints of the stones, but in many harbor works, the jet of water enters freely into the interior of the masonry and introduces such complexity into the question as to render it impossible, at least in the present imperfect state of our information, to give any rules for directing the engineer. I can therefore simply point out the nature of the different forces which enter into the question of the stability of such loosely built structures, so as to show in what direction the sea attacks the works.

"The impact of the waves against the outside of a sea wall or pier gives rise to four distinct forces, viz.: First, the direct horizontal force which tends to shake loose or drive in the blocks of which the masonry consists; second, the vertical force acting upwards on any projecting stone or protuberance, as well as against the lying beds of the stones; third, the vertical force acting downwards, which results either from the wave breaking upon the toe of the talus wall or from its passing over the parapet and falling upon the pitching behind, so

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as to plough it up ; fourth, the back-draught which tends by reaction from the wall to remove the soft bottom, and in this way to undermine the lower courses of the work."

Again he says :

"Within the masonry, as well as without, the waves exert forces in the following different ways: First, by propulsion of vibrations produced by the shock of the waves on the outer or sea wall through the body of the pier to the inner or quay wall. Second, by the direct communication of the impulses through the particles of the fluid occupying the interstices of the hearting so as to act against the back joints of the face wall of the quay. Third, by the sudden condensations and expansions of the air in the hearting so as to loosen and at last to blow out the face stone of the quay, combined with, fourth, the hydrostatic pressure of the water which is forced through the sea wall and from want of free exit is retained and acts as a head at the back of the quay and which, however small in quantity, will act as in a Bramah press upon all surfaces exposed to its pressure, however great those surfaces may be. The three last causes are probably the most efficient agents in the work of destruction."

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DISCUSSION.

CHARLES MACDONALD, M. Am. Soc. C. E.—The effect of waves is well shown in the experience at the breakwater at Havre, France. The exposed part of this breakwater is curved like that of the front of a masonry dam of large size. When the breakwater was finished it was found that the waves passed over the top and into the basin beyond, to the serious inconvenience of vessels. This action was reduced by dumping boulders at the toe of the sea face of the wall. The energy of the waves was spent in rolling the boulders upward. The plan is still followed, a certain amount of stone being dumped into the sea at the front of the wall every year to replace that ground to sand and washed away. Does not this indicate a decided frictional reaction? Mr. Macdonald

CORRESPONDENCE.

RUDOLPH HERING, M. Am. Soc. C. E.—The force of waves acting vertically upward should undoubtedly be given consideration, although in most cases it may be small compared to other forces, and furnish merely a reason for avoiding projections from the face of structures exposed to wave action. Mr. Stevenson, see page 157 of the paper, has considered its action against "any projecting stone or protuberance, as well as against the lying beds of the stones." Mr. Hering.

In speaking of the dynamometer pressures of waves obtained by Mr. Stevenson, the author says that they do not measure the force of a wave, but solely a local pressure which can be produced by it, as only a small portion of the water in the wave is projected to the recorded heights. He therefore thinks that the high pressures should be considered in the details of the work, but questions the necessity of taking such pressures as a measure of the effect upon a large surface of masonry. The writer does not think this conclusion sufficiently definite, nor can he see the identity of the parallel drawn between wave pressure and the pressure of an engine wheel on a bridge. The latter consists of a framework, with different duties assigned to each member, while in a sea wall there is a comparatively uniform, homogeneous and solid mass, with the same cross-section at every part, substantially even when buttresses are added. If every part of a wall is made strong enough to resist these enormous isolated pressures, why, then, should not the whole wall be practically strong enough to resist them, even if extending over a large surface, and to what area of wall would the author have this action applied, when he says that at the Scilly Islands, "large masses of water have been driven over the summit of this rocky island, which is 130 ft. in height"? The press-

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Mr. Hering. ure of sea water 130 ft. in height is 8 340 lbs. per square foot, or more than Stevenson's greatest dynamometer measurement on small disks at Dunbar, which was 7 840 lbs. To drive "large masses" of water over such an island means a large mass at the bottom, and to drive water up an inclined rocky slope to such a height requires a still greater force at its foot than that mentioned. It seems necessary to the writer to take up each case of this kind by itself, and to consider it in the light of all forces liable to act, instead of generalizing further than to find out what simple mechanical principles apply.

The writer does not consider the division of free waves into "long" waves, "oscillating" waves and "ripples" a very happy one. Oscillating waves, particularly in the ocean, can be very much longer than the so-called "long" waves in very shallow water, or near the shore. Ripples, the author explains, run by surface tension. So far as the writer's experience goes, ripples are waves generally caused by the friction of air moving over the surface of water, like most other waves. They can be seen plainly when a slight gust of wind starts over a perfectly calm sheet of water, as in a pond. The writer has also seen them under the same conditions on the ocean after a day's calm. They are the first result after wind commences. If the wind continues and blows stronger, then another and larger set of waves is generated, upon the surface of which the ripples still continue, though less regular in form. If the wind blows still stronger, still larger waves set in, carrying the two other kinds of waves on their surface. When crossing the Atlantic, the writer could, on a certain occasion, distinctly discern four classes. On the Jersey shore, perhaps twelve hours after a storm, when watching the surf, he once distinctly noticed that about every seventh to tenth breaker would run further up the beach than the others, and, in addition to the breaking waves, smaller waves or ripples could be seen upon them which had been started by a fresh breeze. It was evident that there still existed the effect of some large waves coming in from the ocean and caused by the previous storm.

On a dry, sandy beach similar effects can also be noticed. What in this case are often called sand ripples are also the effect of the friction of air upon the particles of sand and are not due to surface tension. A flag or streamer waving in a breeze has its wavy motion imparted by the friction of passing air, similar to the case of water waves.

Generally speaking, there seems to be no difference in kind between large and small free waves, but merely a difference of degree. A definite relation exists between the causing force and the character of the material which is set in motion and its freedom to move. The writer had never seen this general problem worked out, but it applies equally to solid, fluid and gaseous bodies moving over each other, and of which there are nine possible cases. Unless the friction is reduced or wholly overcome, as by a lubricant, such movements produce

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oscillations. These may be in the form of sound waves of high pitch, as in some cases of solids moving upon solids, or merely a rattling noise when, for instance, the ferrule of an umbrella is made to slip over the surface of a sidewalk, or these oscillations may be a visible tremor as when water flows through a hose, or be in the form of a spiral motion assumed by bubbles of air when rising through water. Mr. Hering.

Unless the author can offer a better explanation, it seems to the writer that "surface tension" rather explains the somewhat remarkable fact, that oil poured upon highly agitated water will, by hanging together and spreading out into a thin film, actually reduce the oscillation of waves and completely stop the ripples. The water surface does then not as readily yield to the attack of the wind, nor permit the breaking of the waves, the so-called whitecaps. It is trustworthily stated that a teaspoonful of oil will "calm" a surface of 1 000 square meters or about ¼ acre on the ocean. Heavy and thick oils are said to be the most effective.

ROBERT FLETCHER, Assoc. Am. Soc. C. E.—Among the earlier contributions to the literature of civil engineering is an interesting work by Col. A. R. Émy, an officer of the corps of engineers of the French army, entitled: "Du Mouvement des Ondes et des Travaux Hydrauliques Maritimes, Paris, 1831." The author devotes considerable space to disproving the then commonly accepted theory that in wave motion the individual particles of water move up and down by filaments which, through all phases of the displacement, remain vertical. Having shown this theory to be untenable, he then opposes the practice of giving to exposed sea-walls or breakwaters a vertical or slightly battered face, bringing to bear a great array of facts and arguments to show that such structures are assailed by enormous masses of water acting by horizontal impact, and should deflect rather than reflect such masses. Mr. Fletcher.

Col. Émy was recognized by one or two of his contemporaries as among the first to propound the true theory of wave motion, that is, the orbital movement of the individual particles. His treatment of the subject is free from mathematical analysis, although he presents very clear and ingenious geometrical demonstrations; he argues from facts gathered by his own long-continued observations of wave action and some of its results, while he appeals to the common phenomena with which even less diligent observers are familiar.

Figs, 7, 8 and 9 are drawn on the assumption that the velocity of the particle in the orbit is constant during a complete oscillation, but with the depth of trough exaggerated in proportion to the length of wave. Émy claims that the orbit cannot be circular because the wave profile would then be a curve of sines, and the most cursory observation shows that this is not the fact. He claimed also that the elliptical orbit with minor axis vertical would give a form equally at vari-

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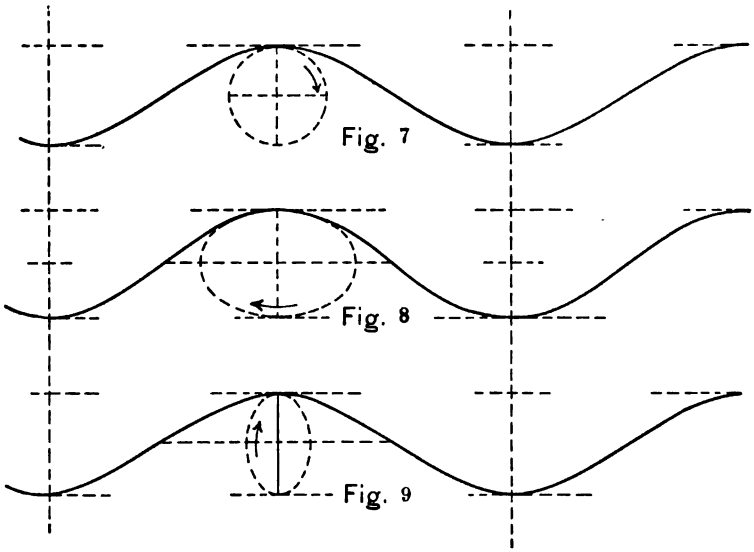
Mr. Fletcher. ance with actual appearance; but that the elliptical orbit with major axis vertical accounts for all actual circumstances of form and motion.

That pure wave motion in deep mid-ocean consists in the motion of the individual particles in elliptical orbits, and that the size of orbit decreases as the depth increases, until perfect calm exists, is now the commonly accepted theory. Émy laid chief emphasis, however, on his conception and definition of the term *flot de fond* or "bottom wave." He says: "I have discovered that the true bottom waves constitute a very important phenomenon of which the nature has not hitherto been recognized, although some of its effects have been noted."

When waves approach the shore from the ideal position in deep mid-ocean and feel the influence of bottom and projecting lands, their



behavior is no longer fully explained by the purely orbital motion of the molecules of water. The theory advanced is chiefly concerned with the effects that the bottom exercises upon the waves. This influence of the bottom is claimed to be of three kinds.

First.—When the general depth is small and the waves are large, they will be shortened and raised, having a more pointed shape due to the reaction of the bottom, causing the vertical axis of the ellipse to be lengthened.

Second.—If the general depth of the sea or ocean is great, and diminishes towards the shore by very gentle and uniform slopes, the undulations are destroyed by gradually stopping the lower particles in their orbits, and also modifying the undulations of the surface particles.

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Third.—The third action is caused, in the ideal and extreme case, by sudden rises or steps in the bottom, and is of the greatest importance. Referring to Fig. 10, let *O R* represent the surface of the calm sea, the curve *B C D*, etc., a vertical section of the wave surface, and *P Q* the level where no movement exists, so that the bottom below this level has no influence upon the wave motion. Let *U T* be a sharp rise, followed by the nearly horizontal bottom *T S*. The undulation *b c d e*, etc., is tangent to *T S*, and it is evident that all the particles of water above this curve will have sufficient space in which to describe their orbits, but that all particles below the curve will lack the room required for a full undulation. The latter particles form under each wave of the undulating mass bodies of water limited by the curve and the

Mr. Fletcher.

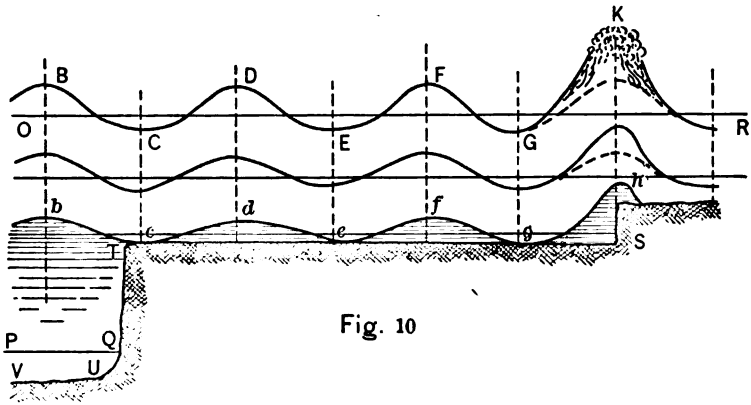

Fig. 10

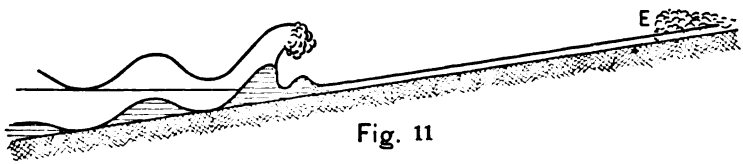

Fig. 11

bottom. These entrained masses of water with profiles of the general shape *c d e* are subject to pressure on the rear, for the wave above is descending to form the trough, while it is relatively relieved from pressure in front, for the water above is rising to form the crest of the advancing wave. By the purely oscillatory action of the wave above, or surface wave, the entrained mass must travel in the direction of the wave motion and with the same velocity. Each oscillatory and apparently advancing wave* entrains a new body of water, and these masses

* In the normal or oscillatory wave no particle of water has a greater horizontal displacement than the length of the horizontal axis of its orbit. Any person who has attentively observed a floating object tossed by waves where there is no current has recognized this as an elementary and fundamental fact. It is illustrated by the waves which sweep over a field of ripened grain and by familiar experiments in the physical laboratory.

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Mr. Fletcher. of water, forced along the bottom under the normal waves, are defined as "bottom waves." Here is a true wave of translation, which has little or nothing of an oscillatory character, and which must exert horizontal energy nearly in proportion to its mass and the square of its horizontal velocity.

When the bottom rises in a series of slight but abrupt ascents, the bottom wave formed at the first rise is forced to mount those succeeding by the pressure from the wave above it, and, as each rise causes a bottom wave on its own account, the bottom waves increase in volume as they proceed, and may carry a large mass of water towards the shore. If such waves meet an inclined plane after passing the sharp rises, they are retarded by the bottom, and, as the pressure continues with little diminution, they are forced into shorter and higher waves, which cause the surface waves to change in form similarly. The energy required to give the bottom waves their velocity is furnished by the normal or surface waves, and this loss of energy causes the latter to shorten and lose velocity. The effects of the incline may be likened to those caused by a succession of small rises. Fig. 11 may represent the appearance at the instant when the bottom wave has become of such a form as to break the surface wave and run up on the shore. The sudden projection of the surf upon the inclined shore is indicated at *E*, Fig. 11. If the velocity and volume of the bottom wave are great, it may break the surface wave when it encounters a rise in the bottom, as at *K*, Fig. 10, causing foam and spray as each wave passes this point.

Often this breaking of the waves over a submerged bank destroys the undulation to such an extent that the sea is quite calm after passing the line of the surf. The pressure upon the bottom wave may not be sufficient to bring any part of it near the surface; but its influence will always be shown by some change in the character of the surface waves, since such a fluid mass is very sensitive to any expenditure of energy. The bottom waves, formed and increased in volume by successive shoaling of the water, are hurled against steep shores or artificial breakwaters with great force.

Such is Émy's theory of the bottom wave. Being literally outside of the limits of the mathematical wave of oscillation, its existence and action might not be considered demonstrable by pure analysis, but the hypothesis does not contravene physical laws. Later authors, English and Italian, cite observations and draw conclusions as to the proper profile of sea-wall sections which are in exact accordance with the doctrine and practice which Émy developed from this theory. It is certain that any mathematical theory of wave action rests upon ideal conditions quite as extreme and unusual as those upon which the theory of the bottom wave is based. The simple, unmodified undulation of the surface of the sea, showing the ideal billow, is not to be seen every day. The great swell of the ocean presents it on a large scale, but

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usually there are primary, secondary and tertiary systems, one superimposed on the other, one from distant and another from local sources of disturbance, with modifications, so that the resultant action is often apparently confused, irregular and interrupted. Hence the engineer desiring a proper working knowledge as to the principles of construction of maritime works must study chiefly facts, phenomena and the accumulated experience of his predecessors in this line. Some facts and phenomena in addition to those cited in the paper will now be presented. Mr. Fletcher.

Prof. Airy[1] asserts that the horizontal motion of the particles of water next to the bottom, produced in shoal water by long waves, is proved by the sensible disturbance of the sand and stones of the bottom; and that waves break over the edge of the Bank of Newfoundland when the waves are high, although the depth of water on the shallow side of the bank is 500 ft. He also mentions that off the coast of the British Isles, where the best charts show a sudden falling off of the bottom, a similar breaking of the waves occurs.

Similar action is noted by Mr. Kiddle[2]. Speaking of the general belief that the influence of heavy waves ceases at a few fathoms beneath the surface, where all is quiet, he says that swells show marked increase in height when they roll over the edge of soundings. "On the parallel of Cape Clear, in longitude 15° W., seamen are familiar with this phenomenon, although the depth is nearly 500 fathoms. At times it is so marked that the dead reckoning may be checked by carefully noting the increase in the depths of the hollows of the waves." "Shortly after the edge of the sounding is passed, the sea becomes more regular and less dangerous to vessels."

The great depth to which the action of waves may be felt is illustrated by a common occurrence off Nantucket Shoals, noted by Mr. Kiddle. Pilots and masters of vessels assert that sand is frequently left on deck after a sea has broken on board, although the depth of water is from 75 to 90 ft. The waves of the Pacific off the western coast of the United States are described by observers as much greater than those of the Atlantic, and many record their breaking in 15 fathoms of water.[3] Charts[4] show the bottom to be in general more precipitous than the bottom off the eastern coast.

In the opinion of Mr. Michael Scott[5]:

When waves come upon a slope the vertical oscillation is converted into a motion parallel to the slope. Surf waves are greatest when deep water shoals suddenly. When the water shoals the waves rear up, change their profile and have a progressive motion imparted to them.

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[1] "Encyclopedia Metropolitana." Article, "Tides and Waves."

[2] "Nature," December 9th, 1875.

[3] Report of Chief of Engineers, U. S. Army, 1881, p. 2635.

[4] U. S. Coast Survey Charts, Nos. 676 and 681.

[5] *Proceedings* of the Institution of Civil Engineers, Vol. xviii, pp. 75, 76.

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Mr. Fletcher. Mr. Cockburn Curtis, who had extensive practice on coast surveys under the British Admiralty in all parts of the world, in a discussion on the form of breakwaters,[1] said that much might be done toward determining the best form and materials for their construction by a careful observation of the character of the waves and of the form of bottom over which they approach the shore. He found that when the general configuration was decidedly steep at the shore line, there was very little change in the waves; but where great differences of level, or steps, existed in the profile, what might be called shoal waves or surf waves were generated. Mr. Curtis decided from his study of the surf waves on the west coast of Central America and on the coast of Coromandel, where he believed the heaviest surf to exist, that the waves were generally greatest where, in approaching the shore, the deep water was suddenly succeeded by a long series of shoals. "The ocean or deep-water wave, when it came in contact with the first shoal water, immediately appeared to rear up or increase in height and subsequently to alter its profile so that the mass of the water was thrown into the front part of the wave and a progressive motion was communicated to the particles of water. * * * The consequence was that the wave always took up with it a mass of water along the slope."

The author of an article entitled, "Notes on the Effects of the Waves Upon the Neapolitan Shore, in Connection with the New Coast Works," gives[2] the following conclusions from his observations:

"Whatever theory may be entertained as to the origin of waves, this fact is undisputed, that near the shore the vertical motion is transformed, as the depth diminishes, into a more or less rapid horizontal movement of translation, and that, by reason of this transformation, the waves acquire a horizontal action upon shallow shores which at times is very energetic, causing them to break violently against any obstacle they may encounter. The translation may be partial or complete; in the first case, they are 'bottom waves'; in the second the water has become shallow enough to cause the 'bottom wave' to be the greater part of the whole wave, which causes the entire mass of water to be thrown forward."

The following facts are well known: When a body of slightly greater specific gravity than the water is thrown off from the beach it sinks, but is returned to the shore very soon. Debris and portions of the cargo from wrecked vessels, whether capable of floating or not, are driven ashore. Dead bodies are washed up in the same manner (by the bottom waves?). A remarkable instance of this action is given by Sir John Robinson.[3] At Madras, during a violent storm a quantity of pig lead was cast on the beach, and it was proved that it came from a vessel which had been wrecked more than a mile off shore.

Mr. John Murray,[4] when working in a diving bell at Sunderland, in 20 ft. of water, found it impossible to work on one occasion because

[1] *Proceedings* of the Institution of Civil Engineers, Vol. vi, p. 125.

[2] *L'Ingegneria Civile e l'Arti Industriali*, 1885, p. 53.

[3] *Proceedings* of the Institution of Civil Engineers, Vol. ii, p. 128.

[4] *Proceedings* of the Institution of Civil Engineers, Vol. xix, page 669.

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Mr. Fletcher.

of the great agitation of the water at the bottom, although at the surface there was but a slight swell. This is a direct proof of the movement of translation at the bottom, while the surface presented only the movement of oscillation.

Smeaton's account of the tremendous force of the waves at Eddystone reads:[1] "The waves would strike the rock and house and fly up in a white column, enveloping it like a sheet, rising to at least twice its height (about 200 ft.) and totally intercepting it from sight." Smeaton believed the cause of this action to be due to the position of the reefs and the shape of the bottom rock. "A circumstance which tends to augment the force and height of the sea so breaking upon these rocks is their particular form and position. * * * They not only stretch across the channel in a north and south position to the length of about 100 fathoms; but they lie in a sloping manner towards the southwest." This sloping continues below the water for a mile or more. "From this configuration it happens that seas coming uncontrolled from the deep water, and, rather suddenly at last, though gradually meeting the slope of the rocky bottom, they are swelled to a degree, in storms and hard gales of wind, as to break upon the rocks with most dreadful violence." The chart of the locality shows that the reefs form a gradually rising trough or gorge, doubtless broken by steps and ridges, leading up to the level where the stone tower was built, from the direction of the heaviest seas. "I particularly noticed the manner in which the waves began to gather, as soon as they came so near the house as to be sensible to the sloping rocks underneath them. These waves, by degrees towering higher as they came nearer, formed a deep hollow sea at the foot of the building; and then, falling into it, struck it with all imaginable fury."

The late Thomas Stevenson, referring to the wave action at Bell Rock Lighthouse, quoted the late Robert Stevenson as follows:[2] "The dispersion of fishes, by their disappearance from the fishing grounds in stormy weather, tends to show the disturbance of the depth of the ocean at the depth of 30 or 40 fathoms. * * * Some drift stones of large dimensions, measuring upwards of 30 cu. ft., or more than 2 tons in weight, have, during storms, been thrown up from deep waters. These stones are so familiar to the lighthouse keepers at this station as to be by them termed travelers."

It appears from these citations that a motion of translation of portions of the wave when it reaches shallow water is now generally admitted; also that where abrupt rises in the bottom are known to exist, there the phenomena are observed which Émy's theory describes and explains.

Thomas Stevenson had no records of pressures near the bottom. At Skerryvore Rocks the high and low dynamometers were not placed

[1] John Smeaton's "Narrative of the Building and a Description of the Construction of the Eddystone Lighthouse," pp. 10 and 175.

[2] "Design and Construction of Harbors," Edinburgh, 1874, p. 17.

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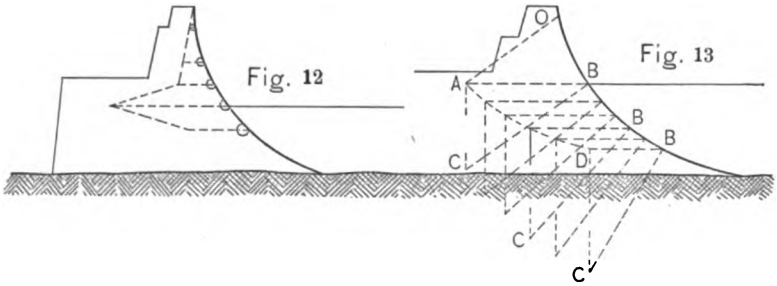
Mr. Fletcher. upon the same rock, but one was 40 ft. seaward of the other, and hence subject to different conditions. At Dunbar flush dynamometers were placed over quite an extent of the curved wall, as shown in Fig. 12. The dotted lines show the variation of horizontal pressure thought to exist. Stevenson gave the diagram as a rough approximation only, as some of the instruments failed.

The following argument is by Mr. A. W. French, the writer's colleague: Masses of water hurled against the foot of a curved wall would be deflected from their horizontal course and would exercise a nearly constant normal pressure on the surface. Of course the horizontal pressure would increase as it passed up the curve. In Fig. 13, with a profile similar to that of Fig. 12, the change in horizontal pressure due to the reaction of deflection, or centrifugal force $B\,C = \frac{W v^2}{g R}$ (w is the weight of a cubic foot of the water, and R is the radius of curvature) is represented by the broken line $A\,D\,B$. The energy of the moving and lifted water would be expended against the force of gravity and would become zero at some point on or above the wall. Some



line, as $A\,O$, connecting A with the point of zero energy, would complete the line of pressure.

The similarity of Figs. 12 and 13 is apparent. Stevenson's figure shows a sharper diagram, but it is reasonable to suppose that this is due to the impulse of the surface particles as they are suddenly stopped in their orbits by the wall. This impulse would lengthen the line $A\,B$ and make the agreement still closer.

At Dunbar the action upon the unfinished wall was noteworthy. Dynamometers were attached to piles a few feet in front of the wall and also on the wall itself and pressures noted. Stevenson remarks:

"These experiments prove, therefore, that oscillatory waves become waves of translation when they reach the unfinished part of a vertical sea wall, and that they then exert a force nearly six times greater than if they had remained waves of oscillation. It further appears that purely oscillatory waves do not exert much more than their hydrostatic pressure, under conditions similar to those affecting the Dunbar experiments; but had there been a storm of wind, those waves would no doubt have ceased to be purely oscillatory, even though the water had been deep."

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Mr. Fletcher.

Referring to the Skerryvore experiments, assume the maximum dimensions of waves to be 600 ft. in length, 43 ft. in height, with a velocity of 50 ft. per second.[1] The velocity[2] of the surface particles in their orbits would be approximately 12 ft. per second.

$$\frac{\text{Mean speed of particle}}{\text{speed of wave}} = \frac{\text{Circumference of orbit}}{\text{length of wave}}$$

The pressure of the water striking the dynamometer at this velocity would be about 290 lbs. per square foot $\left(p = \frac{Wv^2}{g}\right)$. This pressure would decrease from the surface downward. The static pressure of the wave at the sea level would be $43 \times 64.4 = 2\,769$ lbs. per square foot, or an average pressure of 1 385 lbs. If the average pressure due to the impulse of the particles be $290 \div 2 = 145$ lbs., the combined pressure would be only 1 530 lbs. per square foot. But the maximum force recorded was above 6 000 lbs. If, now, some of the water is assumed to move horizontally with the wave velocity of 50 ft. per second, then the force of the impact will be theoretically $\frac{64.4 \times (50)^2}{32.2} =$ 5 000 lbs.; if 1 385 lbs. (neglecting 145 lbs., since the particles of the bottom wave have no orbital motion) is added to this, there results 6 385 lbs., which is a good agreement with the experimental results.

Mr. Cooper.

THEODORE COOPER, M. Am. Soc. C. E.—No modern authority on waves holds to that part of Col. Émy's theory which requires that the major axis of the ellipse be vertical. The special cases of wave action mentioned by Prof. Fletcher and the phenomena, for the explanation of which the bottom wave theory was advanced, are all covered sufficiently for the purpose of this paper by the general statement, "the off-shore waves running into shallower water, or converging into narrower limits by the form of the coast or bottom, will impart their energy to smaller masses of water, producing waves exceeding those originating them." While the manner in which waves are formed and modified and their action under various conditions are instructive and full of interest, the important points, from an engineering point of view, are the character of the waves that will reach any proposed structure and the force they will exert against the work. The height of any wave, regardless of the manner of its generation, is the measure of its energy, and therefore the measure of the force necessary to resist such wave, according to the method to be employed, whether to reflect, divert or destroy it.

The computations at the end of Mr. Fletcher's remarks are incorrect. The orbital energy and the static pressure cannot be added, for the static pressure is produced by the orbital energy. Neither can the impact of a wave be determined by use of an impact formula, nor is

[1] Stevenson's "Design and Construction of Harbors," p. 35.
Rankine's "Civil Engineering," p. 754.

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Mr. Cooper. it necessary, for the height of the wave measures its energy, and twice this height gives the greatest measure of the pressure needed to reflect it. His assumption that some of the water may move with a velocity equal to the wave velocity is untenable, for such particles could not continue in the wave, but must fly off by centrifugal force. If such an assumption were applied to the Atlantic tidal wave, there would be twice in 24 hours' pressures along the whole seaboard of 1 500 000 lbs. per square foot.

Mr. Hering asks: "If every part of a wall is made strong enough to resist these enormous pressures, why, then, should not the whole wall be practically strong enough to resist them"? Walls are not the only form of sea structures. In a solid wall the joints and projections would be the parts most seriously effected by these enormous pressures acting over only limited areas. In timber crib walls or bulkheads the detail parts might be entirely too weak to resist the localized pressures, and yet the whole crib, unless destroyed in detail, be abundant to resist the onslaught of the waves. Lighthouses which were perfectly secure as to the main structure could be and have been seriously effected in their upper works by these limited but heavy forces.

Mr. Hering does not think the division of waves as accepted by modern writers a happy one, and especially objects to "ripples" or that class of small waves which run mainly by surface tension. Surface tension enters into all the problems of liquids, but usually is a very minor factor. In large waves its effects are unimportant, but for minute waves it becomes the principal factor and modifies and in some cases reverses the laws governing large waves. Ripples run the faster the shorter they are, while waves do the reverse of this.

Any one who desires to study into this interesting subject will find "The Construction of Harbors," by Thomas Stevenson; Tides and Waves in the "Encyclopedia Metropolitana"; "Waves and Wave Theory," in the "Encyclopedia Britannica," good material to commence on. All of these have been freely drawn upon in preparation of the foregoing paper.

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No. 788.

A WATER POWER AND COMPRESSED AIR TRANSMISSION PLANT FOR THE NORTH STAR MINING COMPANY, GRASS VALLEY, CAL.

By ARTHUR DE WINT FOOTE, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, 1896.

WITH DISCUSSION.

Upon the prohibition of placer mining by the State of California, the immense canal systems extending over the western slopes of the Sierra Nevadas were left without a purpose, and their future existence depended upon a new use for water. Out of this necessity has grown a business of selling water for power and irrigation, retaining the original methods of delivery at the bank of the canal and miner's inch measurement. The price of water is approximately 1 cent per 1 000 galls. delivered at the canal; its cost for power depends upon the pressure that can be obtained from it. In the case of the North Star plant, it could have been conveyed directly to the mines and have done its work there on different wheels more or less adapted to the varying conditions; but there is a certain inconvenience and danger in using water in this manner under a high pressure, and, moreover, the mines are on a

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hill. So it therefore seemed advisable to convey the water directly to the lowest convenient point, obtain the power there, and transmit this power to the places where it was needed.

This brought forward the subject of transmission of power, and electricity was naturally suggested first. Visits to mines in operation and careful study and investigation of electrical appliances for underground work, especially pumping, finally decided the author in favor of compressed air. The latter method, under the conditions, was believed to be most economical of power, least liable to accident, and cheapest in first cost. Moreover, almost absolute security against stoppage could be obtained by having a set of boilers on hand ready for firing up in case the water power or air plant gave out, for, by the use of these boilers and opening and shutting a few valves, all the air motors become equally good steam motors; whereas, with electrical transmission an entire set of steam motors would have to be provided to give equal security; or, as the air and steam motors are the same, the electrical motors would require just so much extra expense in cost of plant of equal security against stoppage.

To give as briefly as possible a distinct idea of this work, a concise description of the whole plant is first submitted, followed by the detailed presentation of its several parts, and, finally, the results obtained after careful tests during three months of actual working.

The water supply is obtained from the South Yuba Water Company, at a point on their canal about 4 miles from Grass Valley, Nevada County, Cal. Thence it is conveyed about 2½ miles to the Empire Mining Company's works in a 22-in. riveted iron pipe, built more than ten years ago. The new conduit is a riveted steel pipe, 20 ins. diameter, joined to the lower end of this old one under a head of 420 ft., and continues 7 070 ft. to the power house, situated at the lowest convenient point on Wolf Creek, just below the town of Grass Valley, where a head of 775 ft., or a static pressure of 335 lbs. per square inch, is obtained. The capacity of this pipe is sufficient to develop 800 to 1 000 H. P.

At the power house there is a Pelton water wheel 18 ft. 6 ins. diameter, running on a 10-in. shaft, to which a duplex compound air compressor is connected directly. The initial cylinders are 18 ins., and the second cylinders are 10 ins. in diameter, with a 24-in. stroke. They were designed to run at 110 revolutions per minute, and require 283 H. P. from the water wheel.

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A 6-in. lap-welded pipe conveys the air at 90 lbs. pressure from the power house to the company's Stockbridge shaft on Massachusetts Hill, 800 ft. distant and 125 ft. higher. Here it is now being used in a 100-H. P. cross-compound Corliss pneumatic hoisting engine, and a 75-H. P. compound pump, besides other pumps, blacksmith forge, drills, etc,

The Pipe Line.—The line of the pipe is quite crooked, both horizontally and vertically, partly because it was necessary in locating it to follow a county road. The trench was dug with plows and scrapers, except where too stony, and the joint holes were dug by hand. The joint holes cost fully as much as the trench, for the reason that many of them reached down to the harder and rockier stratum below the soft surface material. The trench was about 10 ft. wide on top, 4 to 5 ft. on the bottom, and 4 ft. deep. The joint holes were 4 to 5 ft. long and 3 ft. deeper than the trench. The total cost of all the work of burying the pipe, including covering a large portion of it with stone from the mine dumps, and cement masonry wells for the valves and for sustaining the pipes around bends, amounted to approximately $6 756.27. This was done on company account, after refusing bids, the lowest of which would have amounted to about $8 500.

An aqueduct of cement masonry across Wolf Creek at the power house is not included in this estimate, but was built on company account. As shown in Plate V, Figs. 1 and 2, it was first built up to grade; the pipe was then laid upon it and afterward covered, so that the pipe is now in the center at the tops of the arches, where the masonry section is 4 ft. square. The piers of this aqueduct are carried down to the bed-rock of the creek some 8 or 10 ft. below its water level. The lower portion of the center piers was built up of cement concrete, but the remainder of the bridge is built of the rough stone hauled from the mine dumps in the vicinity, and Portland cement and sand mixed one to three. Very little hammering was allowed, mortar being cheaper than masons. The center arches are of 33-ft. span, and the other two of 24 ft. and 28 ft., the length of aqueduct over all being 153 ft. The rock and sand each cost 75 cents per yard delivered; common labor, $2.50 per day; masons and foreman, $4 per day. The entire cost of the bridge was $1 435; as it contains about 180 cu. yds., the cost per yard was about $8. It is thought that this will prove cheaper in a few years than any other mode of carrying the pipe across the creek.

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Wood or even iron would be subject to more or less change of form, sufficient to cause leakage in the pipe, but it is believed that the pipe buried in the masonry, when once tight, will never have movement enough to cause leakage. In any case, it is probable that a wooden truss bridge with stone piers would have cost nearly as much as the stone. It should be mentioned, however, that the lumber for centers is not included in the cost in this case as it could be used equally well afterward in the mine.

The steel slabs for the pipe were furnished by the Pennsylvania Steel Company. The Central Mills of Harrisburg rolled the plates, and only seven sheets of the entire lot were rejected by the inspector. The Risdon Iron and Locomotive Works of San Francisco manufactured the pipe from the 48 x 66-in. sheets and laid it complete in lengths of about 28 ft. in the trench under the following schedule, the longitudinal seams being double riveted by hydraulic riveters:

Head in Feet.	Length.	No. B. W. G.	Thickness.	Rivets.	Remarks.
420 to 500	2 320 ft.	9	0.148 in.	$\frac{3}{4}$ in.	Cold riveted
500 to 600	2 110 ft. 6 ins.	8	0.165 in.	$\frac{3}{4}$ in.	"
600 to 700	1 158 ft.	7	0.180 in.	$\frac{7}{8}$ in.	"
700 to 750	1 204 ft.	6	0.203 in.	$\frac{7}{8}$ in.	Hot riveted
750 to 775	285 ft.	5	0.220 in.	$\frac{7}{8}$ in.	"
Receiver	40 ft.	.....	0.375 in.	$\frac{15}{16}$ in.	"

The specifications required a mild and very tough steel, and the cold flat bending test was insisted upon for all thicknesses. The pipe was dipped after being made into lengths into the usual hot asphaltum mixture, and then shipped by rail to Grass Valley and delivered along the trench in wagons. It was then rolled into the trench, and the lengths riveted together in place. Where there was no change in direction a slip joint was made by raising the outer end of the length with a small hand derrick, slipping the upper side into the completed portion, and catching it there through the rivet holes with bolts. Then by lowering the outer end, the weight forced the length into its place with a little care and guidance by chisels, when it was bolted ready for the riveters. Where there was a change in direction, bands were put on in two halves, lapping, but no bend piece or band was allowed with more than 4 ins. difference in width between the inside and outside. Where hot rivets were used a $\frac{7}{8}$-in. hole was punched in the top of the pipe near the end, through which the hot rivets were

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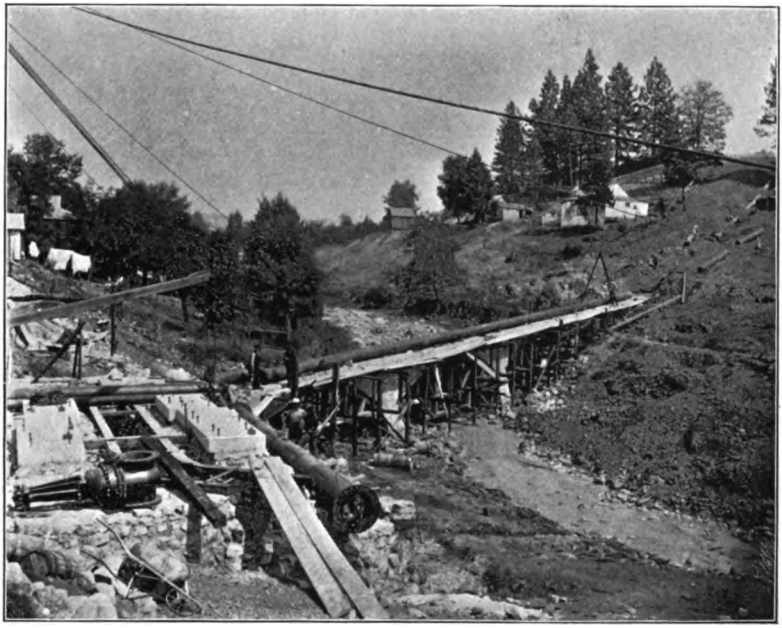

FIG. 1.

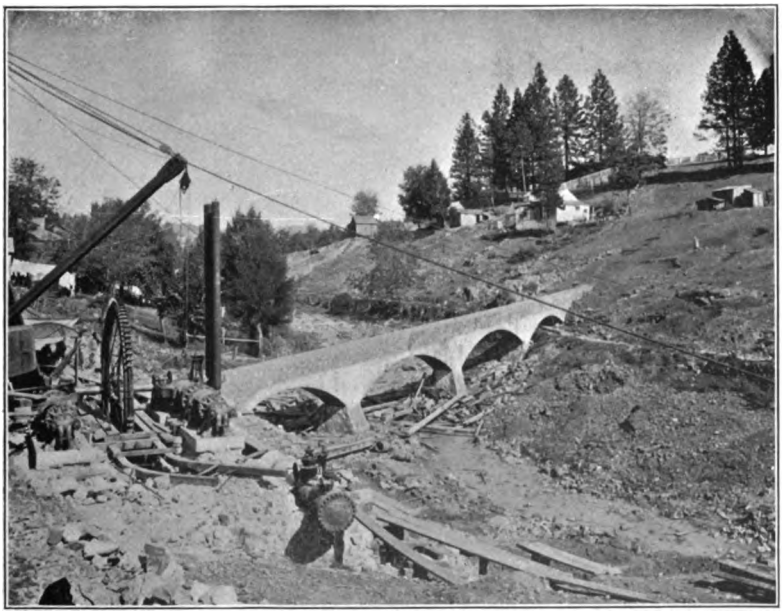

FIG. 2.

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dropped to the holder-on inside. This hole was afterward stopped with a ¾-in. gas-pipe plug. P & B paint was used to cover all points not protected by the original asphaltum mixture, because it was considered preferable to the hot mixture when put on the cold metal.

In filling the pipe with water, it was quite a delicate operation to let on the full pressure at the Empire gate, as it added 180 lbs. pressure per square inch the instant the lower 7 000 ft. became full. It was done several times by keeping open a valve at the lower end of the pipe, while the Empire gate was allowing a little more to pass than the lower valve allowed to run away. In this way the pressure was not raised more than 10 or 15 lbs. above the normal. Numerous very small leaks appeared in the pipe under pressure, but neither then nor since has there been a defective rivet or plate. Fine dust from the wagon road was put into the pipe in considerable quantities, which stopped most of the smaller leaks or sweating. The attempt was made at first to mark the larger leaks, and then take off the pressure and calk them, as in the case of a steam boiler. It was soon found, however, that they would appear again as soon as the pressure was returned. Since the leaks have been calked under pressure, using proper care not to strike heavy blows, all trouble has ceased. For a time, owing to an imperfect gate at the Empire, the pressure was raised 60 to 90 lbs. almost instantly many times a day, and in consequence small leaks developed, though they were easily calked.

Close watching was necessary, however, as a stream no larger than a hair, if it happened to be turned along or against the pipe, would, with the aid of the sand it washed in, cut the pipe badly in a few hours. In one instance two threads of water, so small that they failed to wet the earth upward to the surface, shot out at right angles to each other, one from the longitudinal seam, and one from the circular seam near their intersection. These streams, striking each other, formed a miniature whirlpool, which bored a hole through the pipe about the size and shape of the point of a lead pencil, letting out a larger stream which soon led to its discovery. Where sharp bends were necessary, the pipe, as laid in the trench, was packed on the outer side of the bend by grouting and cement masonry between the pipe and the side of the trench for the full thickness of the pipe.

About 1 000 ft. from the lower end a 12-in. branch with a gate is put in for possible future use, and adjoining it on the lower side a

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20-in. gate. These are buried in cement masonry, to prevent any movement from thrust when the latter gate is closed, and over all is built a cement masonry gate house 12 ft. square, which answers also for a tool house. At the lower end of the pipe in the power house there is another 20-in. gate, below which is a 12-in. branch leading to the Pelton wheel, and adjoining this is the receiver, 2 ft. in diameter, on which are the air chambers, charging tube and relief valve. The lower end is stopped with a flanged end, bolted on, which can easily be removed for extending the receiver for additional power.

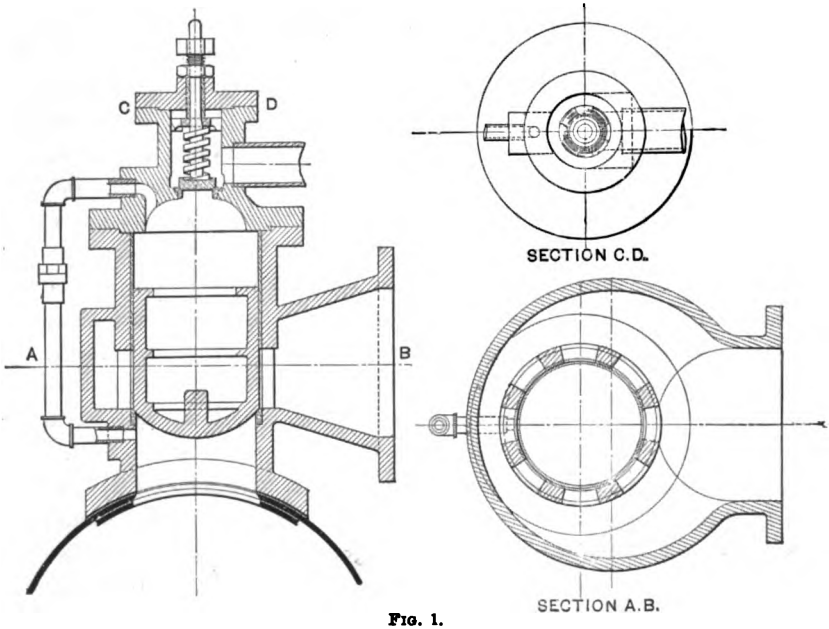

FIG. 1.

The air chamber is a 10-in. lap-welded tube 18 ft. long standing on the receiver, with an 8-in. gate between. The charging tube is similar, but 8 ins. in diameter. Both have 2-in. water discharge pipes and gates, and by proper manipulation of the gates and the operation of inlet check valves on top of the tubes, the air chamber may be filled. Ordinarily the charging-tube is filled up to 90 lbs. pressure from the air compressor delivery pipe, and then raised by the water pressure. It is found necessary to put in about one-tenth of the volume of the air-chamber every day. Where the air goes is, thus far, a mystery, as no leak has been discovered.

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The relief valve was designed by Mr. H. Schussler, Chief Engineer of the Spring Valley Water-Works of San Francisco, and is the first of its kind ever made. It is shown in Fig. 1. It seems perfect for the purpose, except that the small pop-valve leaks continually. It is hoped that this fault may be remedied by using a different form of valve. It can be set to open at about 35 lbs. above normal, and will close without jar or hammer. In action, the pressure rises until it lifts a pop-valve in the ordinary way; when this is raised the pressure is relieved on top of an 8-in. piston, slightly larger at the top end, which rises and opens the ports of an 8-in. outlet. The pressure going down, the pop-valve closes and equalizes the pressure on both ends of the piston, gradually closing it. If the proper pop-valve can be procured, this relief valve will prove a sure safeguard for the pipe. On one occasion already, when from some unknown source a piece of drift wood entered the nozzle and stopped the flow of water instantly, this relief valve saved the pipe from serious shock, if not actual bursting.

Water Wheel.—The demand for direct action under a head of 775 ft. made a large wheel necessary in order to obtain the proper peripheral speed of half the spouting velocity. The manufacturers objected seriously to undertaking anything over 15 ft. in diameter; whereas the proper speed of 60 to 70 revolutions for the compressors required a wheel of nearly 30 ft. diameter. A compromise was finally made with a wheel of 18 ft. 6 ins. diameter, and a compressor revolution of 110 per minute, and the Pelton Water-Wheel Company, of San Francisco, built the wheel from a design by Mr. E. S. Cobb. Had the design been prepared sooner, the wheel could have been made 30 ft. in diameter equally well. The Pelton Company guaranteed an efficiency of 85% at full load, and an average of 75% from half to full load of the theoretical power of the water, and, at the same time, to so govern the wheel that it should not exceed 120 revolutions nor raise the air pressure above 105 lbs. per square inch in case of accident to machinery or sudden shutting off of air. As shown in Plate VI, the rim is built up of angles and plates riveted together to break joints. It weighs about 6 800 lbs., and is held concentric with the shaft by 12 pairs of radial spokes of 1¼-in. rod iron held by nuts to the cast-iron hub. The driving force, being applied to the rim, is transferred to the hub by four pairs of 2-in. iron rods, so arranged as to form a truss, shown

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also in Plate VI. Without going into the calculations for strains, which are given as furnished by Mr. Cobb in Appendix No. 1, it may be stated that the factor of safety is very large for all the strains likely to come on the wheel.

The wheel is set on a 10-in. shaft, having a disk crank on either end connected directly to the compressors. The wheel was easily lined up by the nuts on each end of the radial spokes to run almost perfectly true. After a few weeks' use it was found to have worked slightly and was straightened again. It balanced so well that upon shutting off the water it was 14¼ minutes coming to rest from 110 revolutions per minute, when disconnected from the compressors. The regulator is a floating valve actuated against excessive velocity by the ordinary ball governor and against excessive air pressure by a spring set to move when the air pressure in the delivery pipe exceeds 90 lbs. This floating valve admits water on either end of a hydraulic piston which operates a lever moving a hood up and down over the head of the nozzle, thus shutting off or letting the water on to the wheel as the air pressure becomes too great or as the speed gets above or below 110 revolutions. This regulator has now operated the wheel for several weeks and seems almost human in handling its speed. The load can be thrown off entirely and the governor will hold the wheel to 120 revolutions or less, and if all the air motors happen to be shut down the air pressure will increase rapidly and the wheel slow down, until at 100 lbs. it will stop.

The compressors are so arranged, as will be shown further on, as to admit of being run at one-quarter, a half, three-quarters or full power. It was quite an object, therefore, to have a water wheel which would give as nearly as possible the full efficiency under these different heads. For this purpose there are four nozzles, one for each of the heads required, and, as the machine must be stopped to change its capacity from one load to another, it is no great inconvenience to change nozzles at the same time, requiring perhaps three minutes. It was of considerable interest to know just what the efficiency of so large a wheel might be, and it was also necessary to measure the water quite accurately for business purposes; therefore a measuring flume 18 ft. long and 6 ft. 9 ins. wide was constructed to take the tail water on leaving the building. The overflow is a sharp-edged iron about 3 ins. high and 15 ins. above the bottom of the flume. No contractions are made.

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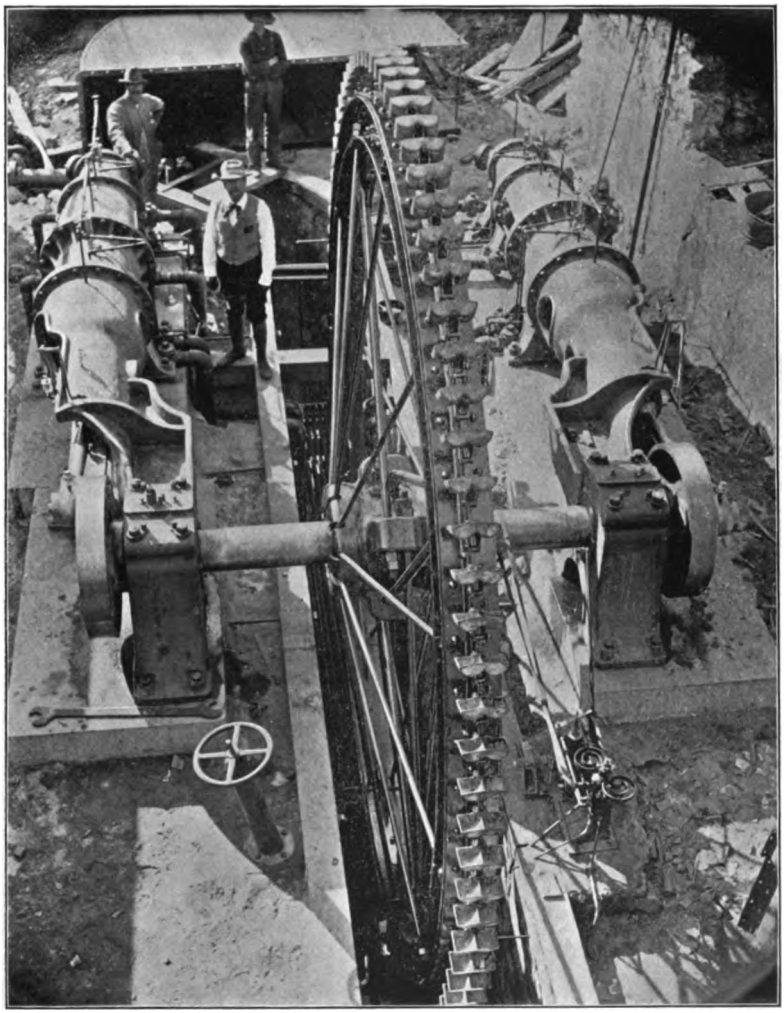
PLATE VI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXVI, No. 788.
FOOTE ON COMPRESSED AIR TRANSMISSION PLANT.



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An 8-in. pipe about 6 ft. above the overflow connects with the gauge box, within which is the measuring scale for the hooked gauge. Great care was used in making the whole apparatus as perfect as possible, and it is believed that the average result of many measurements is within 1% of being correct. The power developed by the wheel was found by taking a large number of indicator cards from the compressors. These were averaged and the friction added, which was found by first running one compressor and afterward running it with the other compressor with the valves out; the difference in the quantity of water used was measured and the horse-power of this water was called two-thirds of the friction of the machine when loaded, which was equivalent to allowing 50% for load. The friction of the water wheel and its bearings is included in its efficiency. Repeated tests which checked very closely give the wheel an efficiency of a trifle over 90% for one-quarter, one-half, three-quarters and full loads. Between these points it is somewhat less, as the hood coming down over the nozzle tends to deflect the water as well as hold it back, and decreases the efficiency. It seems probable that the long radius of the wheel accounts for the high efficiency.

The illustration of the wheel shown in Plate VI was taken before the wheel was tried. The buckets were soon found to be too near together, and one-half of them were taken off. Actual working has shown that this wheel has great efficiency at low speed. It began working under half load, using the half-load nozzle. Gradually in pumping out the mine, as the pumps were lowered and more power was needed, the limit of the machine while running one compressor at 110 to 115 revolutions was reached, and the opposite compressor was connected on, the intention being to run three-quarters load with the three-quarter nozzle. As an experiment, both compressors were connected as if for full load, and the half-load nozzle was retained. The result showed considerable more power delivered with both compressors running at 54 to 65 revolutions than was obtained before with one compressor at 115 to 120, so much, indeed, that the works were kept running over two weeks longer with the half-load nozzle, though the pumps were lowered on an average 18 ins. per day. Of course a large portion of the gain in power can be attributed to the saving in friction and improved working of the compressors under the slow motion, but yet it must be true that the wheel loses very little in its effici-

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ency, compared with the accepted ideas of loss in wheel efficiency, working under a peripheral speed of one-quarter of the spouting velocity.

Compressors.—Mr. E. A. Rix, of San Francisco, who has made a careful study of air compression, designed the compressors shown in Plate VI, and they were built by the Fulton Engineering and Ship-Building Company of San Francisco. They are made very heavy to stand the high piston speed required by the conditions of the water power. Had it been known at the time of designing this plant that a wheel could have been made like the one described, a diameter of 30 ft. instead of 18 ft. would have been chosen and the piston speed of the compressors reduced accordingly, for there is no question but that 60 to 70 revolutions will do better work than 110. The compressor cylinders are 18 and 10 ins. in diameter and 24 ins. stroke, with a water jacket so arranged that two streams of water pass around the cylinders in opposite directions. The inlet valves are of the Brenner pattern and open directly into the power room. Mechanical valves and air inlets from the wheel pit were considered, but taking into consideration the class of mechanics likely to handle the machine, and its remoteness from the shops, it was decided to adopt the simplest valves and place them so as to be reached easily. The compressor cylinders were made sufficiently large to allow for the rarefaction of the air caused by the automatic valve, and all the air that enters the building is forced to come up through the wheel pit. The other valves are made after the pattern of ammonia compressors.

The most novel feature of these machines is the intercooler. This is made up of 49 soft copper pipes, 1 in. in diameter, 18 ft. long, each with a stuffing-box at each end connected with manifold castings. The air delivered from the first cylinder into one manifold passes through these pipes to the other manifold, from which it is taken to the second cylinder. The whole is placed in the wheel pit directly under and in front of the wheel, so that the water dashes all over and through it. The air leaving the first cylinder at a temperature of 200° Fahr., passes through the intercooler and enters the second cylinder at 60°, slightly cooler than when entering the first cylinder. The temperature is again raised to 204° on leaving the second cylinder and passing into the transmission pipe, showing a total rise in temperature of 282° Fahr.

The transmission pipe, conducting the air at 90 to 100 lbs. pressure about 800 ft. from the compressors to works at the mine, is ordinary

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well tubing 5¼ ins. in diameter inside, screwed together in the trench and bent to fit the uneven ground by building fires around it and heating it until its own weight shaped it to the surface. After being laid and covered, it was tested by filling it with water under 120 lbs. pressure, which was allowed to stand over night. No leakage was discovered and none has appeared since. As yet only half of the full load of air has been passed through this pipe, so no data of value regarding loss in transmission through pipes have been obtained. With the present load the difference in pressure between the power house and the mine is not sufficient to be detected on the ordinary pressure gauge. At the mine there is the ordinary air receiver, and also three 50 H. P. boilers set ready for steam, which are used for receivers.

The air is taken from these into the reheaters designed by Mr. Rix and built by the Fulton Company. As these are the first of their kind, it may be said they have proved remarkably well designed for their work. Experience has shown, however, that slight improvements can be made which would save fuel. At present it requires a little over half a cord of good pine wood each 24 hours to heat about 700 cu. ft. of free air per minute to a temperature of 350 to 400° Fahr.

The heated air passes through pipes covered with magnesia and hair-felt to the first cylinder of the hoisting engine, from which it is exhausted back into the upper heater, where its temperature is again brought to 350°, whence it passes to the second cylinder at 30 lbs. pressure. From this it is exhausted through a flue to the change house, where it is used for heating and drying clothes. From the first heater also the air for the pump is conveyed some 300 ft. down the shaft in a similarly covered pipe. The pump was designed and built by Mr. George E. Dow, of San Francisco, and is a tandem compound vertical sinking pump of a capacity of 500 galls. 300 ft. high per minute. It receives the air at about 275° and exhausts it into the shaft at about 60°, thus giving plenty of pure cool air to the men, without the usual fans or ventilators. At the present writing, this pump is throwing 600 galls. per minute 240 ft. high.

In addition there is a direct-acting donkey pump throwing 350 galls. 110 ft. high situated in another shaft 750 ft. distant, to which air is carried cold in a 2-in. pipe over the surface. An old hot-water heater is used as a reheater for the air, and consumes 12 sticks of pine cord wood per 24 hours.

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The hoisting engine is a compound direct-acting Corliss of 100 H. P. with cylinders jacketed for hot air, and is calculated to work 3 000 ft. down an incline of about 35 degrees. This was also designed by Mr. Rix and built by the Fulton Company. While it is especially adapted to the use of heated air, it takes steam as well as any engine and in fact was first tested with steam.

Efficiencies.—Efficiencies often seem to depend largely on the personal equation of the reporter. Mr. Rix's summary of tests is given in Appendix No. 2. He spent a number of weeks making them, and while agreeing with him in the main, the following are submitted as the author's conclusions. In any case, the plain tale is this. There is 304 theoretical H. P. in the water used at the power house, the work actually accomplished at the mine amounts to 203 H. P., and the cost of reheating is $3 per day.

Efficiency of compression and transmission from water wheel to motors, and not including cost of reheating..... $\frac{225.32}{283} = 79.5$ per cent.

Efficiency of compression and transmission from theoretical power of the water to the motors, and not including cost of reheating: $\frac{225.32}{304} = 74$ per cent.

Efficiency from the water wheel to and through the motors, not including reheating..... $\frac{202.7}{283} = 71.6$ per cent.

Efficiency from the theoretical power of the water, to and through the motors, and not including the cost of reheating..... $\frac{202.7}{304} = 66$ per cent.

Efficiency of compression and transmission from water wheel to motors, including the cost of reheating expressed in water power: $\frac{225.32}{307.66} = 73$ per cent.

Efficiency of compression and transmission from the theoretical power of the water to the motors, including the cost of reheating expressed in water power..... ..... $\frac{225.32}{329} = 68.4$ per cent.

Efficiency of compression and transmission from the water wheel to and through motors, including cost of reheating expressed in water power..... $\frac{202.7}{307.66} = 65.5$ per cent.

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Efficiency of compression and transmission from the theoretical power of water to and through the motors, including cost of reheating expressed in water power..... $\frac{202.7}{329} = 61.6$ per cent.

Horse power of air at works after reheating: 225.32.

Horse power delivered to compressors by water wheel: 283.

Theoretical horse power of water used on the wheel: 304.

Horse power of work actually done by the motors: 202.7.

The horse power delivered by the water wheel to the compressor, to which is added the horse power (24.66) which the cost of the wood used in reheating would buy in water: $307.66 = 283 + 24.66$.

The theoretical horse power of the water used on wheel added to the horse power (24.66) which the cost of the wood used in reheating would buy in water: $329 = 304 + 24.66$.

After nearly three months' working of the plant, the author believes the results obtained demonstrate the wisdom of having chosen air instead of electricity.

It may be urged that the conditions are particularly favorable to compressed air, as the transmission is short and the power is not needed for tramways or lighting. For lighting it is admitted without question, and possibly for tramways, that electricity is preferable, but for transmission, were it 20 miles instead of 1 000 feet, it is thought by the author that, taking the whole plant, compressor, transmission pipe and motor, as against generator, transmission wires, transformers, and electric motors, the air will prove cheaper in first cost, higher in efficiency, less liable to accident, and less expensive to operate and maintain.

The sense of being solidly supported in his plans and expenditures and encouraged to make his work thorough by the money power behind him is so rare in the experience of the American engineer as to be worthy of grateful mention; but still more rarely does he find, united to this material backing, the moral support which comes of the comprehension and sympathy of a wise engineer. This was the great good fortune of the author in carrying out the decision in favor of compressed air transmission and other somewhat novel features of the plant, under the sanction of Mr. James D. Hague, the President of the North Star Mining Company.

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APPENDIX No. 1.

THE PROBLEM OF THE WHEEL.

By E. S. Cobb, Esq.

The problem is to design a wheel, the total weight of which is to be 10 000 lbs., as nearly as possible, the diameter 18 ft. 4 ins., the rim of suitable section to receive the buckets of a Pelton water wheel, and the whole strong enough to be in continuous use at 110 revolutions per minute while transmitting 226 H. P. from rim to shaft.

The cross-section of rim adopted is shown in Fig. 2. It is made up as follows:

Two 6 × 4 × ⅜-in. angles, weighing 20 lbs. per foot, 40 lbs. per foot of rim.

Four 4 × 4 × ⅜-in. angles, weighing 15½ lbs. per foot, 62 lbs. per foot of rim.

Center plate ⅜ in. thick, 28 lbs. per foot of rim.

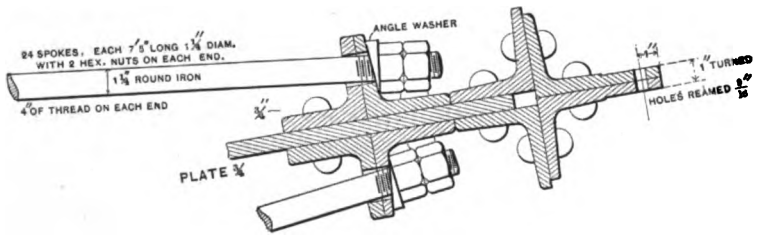

Fig. 2.

The mean diameter of the rim is 16 ft., and the average weight per foot is 135.2 lbs., including rivet heads. The total weight of the rim is 6 800 lbs.

The rim is held concentric with the shaft by twelve pairs of radial spokes. Strictly speaking, however, these spokes are at a slight angle with the plane of revolution, and this angle is taken into account in the calculations.

For the purpose of determining the maximum strain in working that may be expected by one pair of spokes, the rim is assumed to be cut into twelve pieces, each piece weighing 575 lbs., and being held in position against centrifugal force by one pair of spokes. Then the strain on each pair of radial spokes due to the velocity is found as follows:

Centrifugal force $= \frac{\text{weight} \times \text{radius} \times \text{rev.}^2}{2\,936}$. Substituting the proper quantities $\frac{575 \times 8 \times 12\,100}{2\,936} = 18\,958$ lbs., or 9 479 lbs. per spoke.

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By reason of the position of the spoke at an angle with the plane of revolution, this strain becomes 10 000 lbs. per spoke. As 1¼-in. round iron is used, there is at the bottom of threads a safe area of 0.7854 sq. in., or a strain of 12 700 lbs. per square inch of section, on the assumption that these rods hold the whole of the centrifugal force acting in the rim, without any aid from the strength of the rim itself.

The strain at the least cross-section of the rim when it sustains its own centrifugal force without any aid from the radial spokes is as follows:

$$\frac{135.2 \times 8 \times 12\,100}{2\,936} = 4\,458 \text{ lbs.}$$

nearly, per foot of diameter. At 16 ft. average diameter, the total bursting strain of 71 328 lbs. is held by two sections, or 35 664 lbs.

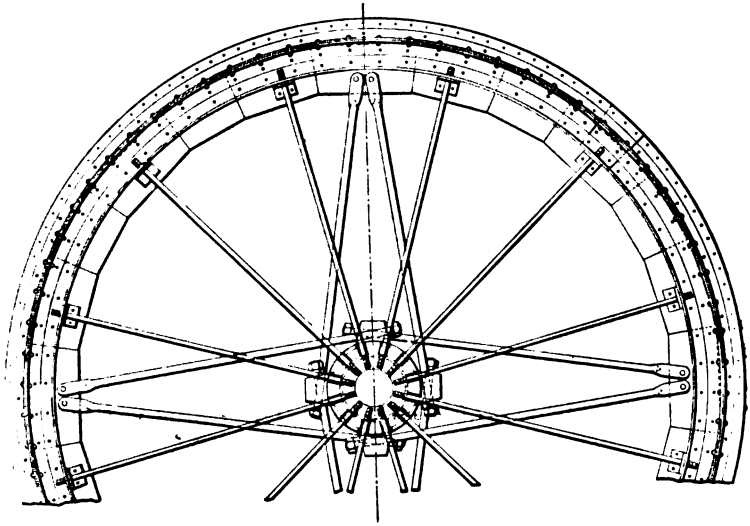

FIG. 3.

per least cross-section. The least cross-section of the rim is the section of the six angle irons, or 30.6 sq. ins. Deducting 25% for unknown imperfections and rivet holes leaves about 23 sq. ins. area, giving a strain of 1 550 lbs. per square inch of rim, when it alone is sustaining all the strain due to centrifugal force.

Power being applied to rim to develop 226 H. P. at 110 revolutions per minute, there results as the tangential strain at the ends of the truss rods 1 500 lbs., or 375 lbs. tangential strain at the end of each truss rod. This tangential strain produces in the rod, on account of its position, a tensile strain of 2 207 lbs., or 1 100 lbs. per square inch of net cross-section when seven threads are allowed for.

The general form of the wheel is shown in Fig. 3.

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APPENDIX No. 2.

SUMMARY OF TESTS OF THE NORTH STAR POWER AND TRANSMISSION PLANT.

By E. A. Rix, Esq.

Actual capacity of compressor, 1 412 cu. ft. of free air compressed to 90 lbs. gauge.

Volumetric efficiency, 96.6%, determined as follows:

Barometer pressure, 13.42. At 90 lbs. gauge, compression ratio is $103.42 \div 13.42 = 7.7$. Receiver, air pipe, and all storage, reduced to 90 lbs. gauge, contained 291 cu. ft., which equals 2 240 cu. ft. of free air. Compressor averaged 102½ revolutions to fill this at 90 lbs. from 25 lbs., which was intercooler pressure. Receiver, etc., at 25 lbs. gauge contained 830 cu. ft. free air. $2\,240 - 830 = 1\,410$ cu. ft. delivered by 102½ revolutions; deducting 2% for increase of temperature in receiver makes 1 382 cu. ft. at 60°, the outside temperature. Theoretical capacity of compressor, 1 429 cu. ft. at 102½ revolutions, $1\,382 \div 1\,429 = 96.6\%$ volumetric efficiency. Reduced to 110 revolutions, this gives 1 412 cu. ft. working capacity. To compress this, 304 theoretical H. P. of water was used on 18-ft. 6-in. Pelton wheel, at 110 revolutions.

The indicated horse power in all the cylinders averaged 250 H. P., and friction load 22 H. P.; total, 272 H. P. $272 \div 304 = 90\%$ efficiency for water wheel.

This friction load was determined when machine had no pressure on it, but was running at normal speed. Experiments on steam compressors show that the friction load is 50% more under full work. This would, then, be $22 + 11 = 33$ H. P. friction load, making total of $272 + 11$, or 283 H. P. delivered by wheel, or $283 \div 304 = 93\%$ of theoretical power in water.

The head of water on the wheel is 730 ft.

The work of compression and delivery is $250 \div 304 = 82.2\%$ of the theoretical horse power, or $250 \div 283 = 88.3\%$ of horse power delivered by wheel.

The 1 412 cu. ft. of air is reheated to 350° Fahr., and, considering that it is used in a compound Corliss engine with intermediate reheating, the theoretical potential in the air before use is 225.82 H. P., determined as follows:

The cylinders being jacketed for hot air the Y which is used in calculations is the same as that used for compound, jacketed compression, viz., 1.3; then

$$Y = 1.3;\ Y - 1 = 0.3;\ \frac{Y-1}{Y} = 0.23;\ \frac{Y}{Y-1} = 4.33.$$

Barometric pressure = 13.42. Outside temperature, 60° Fahr.

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The work being equally distributed between the two cylinders, the intermediate pressure will be the mean proportional between the initial and final pressures. Initial pressure, 90 + 13.42 = 103.42. Final pressure, 13.42 + 1 = 14.42 (1 being the exhaust pressure above atmosphere). Then $\sqrt{103.42 \times 14.42} = 38.62$, absolute intermediate pressure. The formula for the work in the cylinders will then be:

$$\frac{\frac{Y}{Y-1} P V \frac{T_3}{T}\left(1-\frac{p_2}{p_1}\right)\left(\frac{Y-1}{Y}\right)}{33\,000}$$

in which the elements are

$$\frac{Y}{Y-1} = 4.33$$

P = barometric pressure per square foot = 13.42 × 144.
V = cubic feet free air.
T_3 = temperature of reheating, absolute = 461 + 350 = 811.
T = absolute outside temperature = 461 + 60 = 521.
p_2 = intermediate pressure = 38.62.
p_1 = initial pressure = 103.42, and

$$\frac{Y-1}{Y} = 0.23.$$

Then substituting, the equation becomes:

$$\frac{4.33 \times 13.42 \times 144 \times 1\,412 \times \frac{811}{521}\left(1-\frac{38.62}{103.42}\right) 0.23.}{33\,000.}$$

Or 554.96 × 0.203 = 112.66 H. P. in one cylinder, or 225.32 H. P. in two cylinders.

Inasmuch as $3 worth of wood, or its equivalent, 15 ins. of water at 20 cents an inch, has to be added for reheating, the following allowance must be made for this: 15 ins. of water under 730 ft. head equals 30 H. P. The work of compression and delivery being 82% of the theoretical horse power, 30 × 82.2, or 24.66, represents the actual value of power delivered to the air to account for reheating, then, 250 + 24.66 = 274.66 is the total power spent on the air It has a theoretical potential of 225.32 and consequently 225.32 ÷ 274.66 = 82 per cent. Counting from the theoretical power in the water, the efficiency would be 225 ÷ 304 = 74 per cent.

This 82% represents the efficiency of the compressed air system under unfavorable circumstances. It was deemed advisable to have the water wheel on the compressor shaft, and 18½ ft. was the limit of diameter which could be contracted for. This made a revolution speed of 110, and a piston speed of 440, which reached 660 ft. at the half stroke just when discharge was beginning from the cylinders. These piston speeds are too high for economical work. Running the water wheel throttled at 80 revolutions, or a piston speed of 320 ft., the indicator cards showed a horse power of 247, including the friction load. This

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would show an efficiency of $225.32 \div 247 = 90\%$, or from the theoretical water power $225 \div 275 = 80\%$, and were the same plant to be built again this could easily be attained, because it has been ascertained from the experience with the present water wheel that its diameter can be increased so as to run at 80 revolutions.

What power value will be in this 90% efficiency will depend largely on the motors used. The theoretical potential was calculated on the assumption that compound Corliss, jacketed, double-reheated engines should be used. Allowing that these engines have a mechanical efficiency of 90%, which is not a high efficiency, the horse power realized will be 90×90, or 81% of the indicated horse power of the compressors, or $90 \times 80 = 72\%$ of the theoretical value of the water power. With other classes of motors, the value of their work can be proportioned to the above percentages directly as their economic value is to that of the compound Corliss motor from which the air potential was assumed.

When the pump was first started it made 71 strokes, theoretically 710 galls., or 5 893 lbs. The pump has a volumetric efficiency of 95%, which would make this quantity 839 752 foot-pounds. This was done with the compressor running at 80, and the air pressure at 90. At the present time, the pump is making 60 strokes or 4 980 lbs., pumping 230 ft., making 1 145 400 foot-pounds, or, at 95% volumetric efficiency, 1 088 130 foot-pounds. This is being done with 93 single revolutions, at 90 lbs.

The compressor is making 90 single revolutions at an average of 92½ lbs. pressure, which is equivalent to 93 single revolutions at 90 lbs., in order to reduce both the performances to the same pressure.

The difference between the foot-pounds of work in both of these cases, viz., 1 088 130 — 839 752 is 248 378 foot-pounds. The difference in revolutions is 13; consequently 13 revolutions did 248 378 foot-pounds of work, or 19 100 foot-pounds for one single revolution.

It is evident that this work is what the pump is capable of doing, independent of its friction of pipes, etc., because the friction of the pump and the friction of the pipes within the small working limits of this pump are practically the same, as will be seen from the following: For 93 single revolutions, were there no friction of pipes or inertia of pump to overcome, it is evident the pump should perform 93 times 19 100 foot-pounds of work, or 1 776 300 foot-pounds. It really did 1 088 130 foot-pounds, the difference being 688 170 foot-pounds, which is the friction of the pipes and inertia losses.

In the other case 80 revolutions should have done 80 times 19 100, or 1 528 000. It actually did 839 752, the difference being 688 248, again almost identical with the first proposition. So the mean between these two, or 688 209 foot-pounds, is the actual pump loss, and this loss it would seem is almost the same at any head within the working limits of the pump.

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The loss now being determined, it is possible to determine what actual work is being done, and if the pump efficiency is determined, a check can be made on the potential of the compressor.

If at 93 revolutions the pump does 1 088 130 foot-pounds of work, and the friction is 688 248 foot-pounds, and 19 100 foot-pounds are added for each single revolution of the compressor up to its limit of speed, viz., 220 single revolutions, this last amount being 2 425 700 foot-pounds, there would result a total of 4 202 078 foot-pounds of work which the pump would do when the compressor is running its 220 single revolutions, or its limit. If there is deducted from this amount the friction and inertia loss of 688 248 foot-pounds, the result is 3 513 830 foot-pounds, which is the useful effort of the pump, and the ratio between what the pump actually does and the foot-pounds of work it consumes, would be its efficiency, which, upon dividing these sums by each other, shows the mechanical efficiency of the pump to be 83 per cent.

If the potential of the compressor is 225.32 H. P. and the efficiency of the typical motor is 90%, then the brake horse power possible for the typical motor would be 225.32 times 0.9, or 202.7 H. P. for 220 single revolutions of the compressor. For one revolution it would be 0.992, which is 30 426 foot-pounds.

The pump actually does 19 100 foot-pounds for one revolution. The ratio therefore between 19 100 which the pump actually does, and 30 426 which the typical motor would be capable of, would be the pump efficiency as compared to that of the motor. This would be 62 per cent. This includes temperature, the friction in pipes and inertia losses between the reheater and the pump exhaust.

Inasmuch as the pump did 4 202 076 foot-pounds of work with an efficiency of 62%, as compared to the typical motor, the typical motor would yield 4 202 076 ÷ 0.62 = 6 600 000 foot-pounds, or 200 H. P., which compares very favorably with the original calculations for the brake horse power at the potential as being 202.7.

Recapitulating these figures:

The total work done by the pump	= 4 202 076	foot-pounds.
The total brake horse power of the ideal motor	= 6 600 000	"
The total potential of the air after reheating	= 7 434 900	"
The total work spent on the air, including the energy of reheating at 440 ft. piston speed.....	= 9 042 000	"
The total work spent on the air, including the energy of reheating at 320 ft. piston speed	= 8 215 000	"
Theoretical power of the water at 440 ft. piston speed.....	= 10 032 000	"

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Theoretical power of the water at 320 ft. piston speed.....	=	9 205 000 foot-pounds.
Total power of the water, including the energy of reheating at 440 ft. piston speed.....	=	10 692 000 "
Total power of the water, including the energy of reheating at 320 ft. piston speed.....	=	9 865 000 "

Under the lower piston speed, which should be the proper speed to run the compressor, the ratio of original power and reheating, 9 860 000 foot-pounds, and the total pump result, 4 202 000 foot-pounds, would be 44%, which would be the efficiency from first to last as far as the pump system was concerned, and the ratio between any other of the elements in the preceding recapitulation will be their relative percentages.

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DISCUSSION.

S. S. Wheeler, M. Am. Soc. C. E.—While the theoretical power of the water is stated very clearly, there are few figures to indicate exactly the amount of useful work done. The hoisting engine is stated to have a capacity of 100 H. P., but no information is given showing how much work it actually does, and the same remark applies to the other machinery. More definite information on this part of the subject is needed. Mr. Wheeler.

William Doble, Esq.—It is impossible to compare financially the plant described in the paper with one using electricity for the same purpose, since the author has not given any figures of cost of his apparatus. There would have been a saving in the first cost, however, for a 6-ft. water wheel would have been ample to run the generator, and would have cost about one-fifth as much as that installed; the electric generators would have cost less per horse-power than the compressors, and the freight charges on them and the smaller wheel would have been less, and the cost of wiring would have been less than that of the pipes used to convey the compressed air to and into the mines. The cost of installing the electric plant would also have been less, particularly on account of its smaller foundations. The use of covered hot-air pipes in the shafts presents drawbacks which are avoided where electric wires are employed. The pumping plant described in the paper may be compared with one employing electricity which was installed in another mine to handle about 240 galls. of water a minute under a vertical lift of 468 ft. In the latter case, the theoretical power required was 30.1 H. P.; the amount actually delivered was 40 H. P., conducted by two wires instead of large covered hot-air pipes. Another advantage of electrical apparatus over that using compressed air is that it is more easily operated and more economical in operation. The compressor has to be stopped whenever the load varies enough to make it necessary to change the nozzle of the wheels, whereas with an electric generator the varying loads are taken care of automatically in the generator. The guaranteed efficiency with generators at one-quarter load is 85%, and from that condition rises to 95% at full load; higher efficiencies than that can be obtained with air compressors. Mr. Doble.

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CORRESPONDENCE.

Mr. Young. C. G. YOUNG, Assoc. Am. Soc. C. E.—On examining carefully the efficiencies given in the paper, it will be found that only about three-tenths of the power developed by the motors has been transmitted, while seven-tenths has been developed at the reheater. Consequently the plant does not furnish an instance of real power transmission; in fact, hot-air engines without transmission of power from other prime movers are comparable to the case in question and would be nearly as efficient.

It has not yet been demonstrated that compressed air can be transmitted economically for power purposes to any great distance. In most cases it is inconvenient, if not impracticable, to use a reheater at the motor, and unless the reheater is very near the motor, its advantages will be lost. The necessity of such an apparatus will prevent the adoption of compressed air for extensive distribution of power, while the expense of pipe lines and the attendant losses will prevent its adoption for transmission of power to any great distance.

The great loss in air compression can be appreciated when it is considered that all the energy is represented in raising the temperature, and that when the latter falls the difference in temperature represents the energy lost which cannot be regained. Pipes for transmission to any great distance would be very expensive, the allowable pressure would be comparatively low, and the losses in energy would be considerable. The air motor is very inefficient, even when the air is reheated; the reheater is an efficient apparatus, and on this depends the success or failure of a compressed air plant.

In the following comparative table of compressed air and electric transmission of power to a distance of 20 miles, using water power as the source of power, the very high efficiencies given in the paper have been used.

Efficiencies with Compressed Air.		Efficiencies with Electricity.	
Wheel	93%	Wheel	93%
Compressor	88%	Generator	95%
Transmission and reheating	83%	Line and transformer	88%
Total, without motor	68%	Total, without motor	78%

This shows a considerable advantage for electrical transmission. In cases where the reheater cannot be used, the efficiency of compressed air would fall to about 30 per cent. The data given for the electrical apparatus are obtainable in regular practice. The efficiency of the water-wheel appears high, and probably 85% would agree better with the usual conditions; this would lower the total efficiency of air transmission to 62%, and of electrical transmission to 71 per cent. The

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efficiencies given in the paper appear very high, and it is doubtful if they can be maintained in regular practice. Mr. Frank Richards states in his book entitled "Compressed Air," that the net efficiency of modern air compressors in actual practice is never above 80%, and is often below 60 per cent. Assuming 70% as a fair average, the comparison given in the preceding table would be changed to show total efficiencies of 49% and 71% for compressed air and electricity respectively. Mr. Richards claims that it is practically impossible to double the power at the reheater on account of the excessively high temperatures required, and that in the best practice not over 30% is added to the power at the reheater. These statements differ considerably from the author's figures. Mr. Young.

The cost of electric generators and motors would not be more than the cost of compressors, reheaters and air engines, while the copper transmission line would be cheaper than a pipe line and could be erected in places where it would be impossible to lay a pipe line. The cost of a copper transmission line on wooden poles, capable of delivering 10 000 H. P. to the motors 20 miles distant, with a loss of 10%, using 30 000 volts, would be about $35 000. This is about 33 cents per foot. It is evident that a pipe line capable of transmitting 10 000 H. P. a distance of 20 miles, with a loss of 10%, cannot be built at this price. The plant described in the paper is apparently a good example of a compressed air system for transmission a short distance, but an electric plant installed under the same circumstances ought to show better results at a smaller cost. The voltage could be kept low and be suitable for lighting. Electrically operated pumps and drills are fully as efficient as those driven by compressed air, and more portable. No auxiliary set of boilers would be needed in the case of an electric plant, for there would be two electric generators and two or more circuits, so in case of trouble only a portion of the plant would be affected.

The writer desires to ask the author to state the horse-power in the air at the works before reheating, and to compare this with the horse-power at the water-wheel; also to state the pressure, loss in transmission, and size of pipe for delivering 10 000 H. P. to motors 20 miles distant, and the amount of power he would generate at the reheaters in this case.

A. D. Foote, M. Am. Soc. C. E.—Mr. Young states that "only about three-tenths of the power developed by the motors has been transmitted, while seven-tenths has been developed at the reheater." A more careful reading of the paper would have shown him that about seven-tenths of the power is transmitted, and about three-tenths developed at the reheater. Consequently, his remark that this plant does not furnish an instance of real power transmission and his comparison with hot-air engines do not apply. Mr. Foote.

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Mr. Foote. Again, quoting Mr. Young: "The great loss in air compression can be appreciated when it is considered that all the energy is represented in raising the temperature." Were this true, there would be no power left in the air after being compressed and cooled.

Mr. Young says: "The air motor is very inefficient, even when the air is reheated." The reply to this is that the compound Corliss engine is more efficient when working with compressed air than it is with steam, as has been proved by practice, and becomes still more efficient by reheating the air between the cylinders, as is being done in the plant described in the paper.

In the comparative table of efficiencies, as between compressed air and electricity, given by Mr. Young, his efficiencies for electrical transmission may possibly be attained at full load, under ideal conditions; but a transmission plant very seldom is worked at full load, and in air transmission the efficiencies as obtained in working are not perceptibly different, whatever the load, as has been demonstrated by nine months' working of the North Star plant.

Mr. Young presents an average of Mr. Richard's best and worst air compression as a foundation for comparing electric and air transmission. It seems more proper to assume that in putting up a plant the best compressors would be used; and it is proved by work now being done that Mr. Richard's figures are not the last word on air compression.

Mr. Young's statement that the "cost of electric generators and motors would not be more than the cost of compressors, reheaters and air engines," does not agree with the author's experience. The bids from the electrical companies for the North Star plant machinery were considerably more than the actual cost of the air machinery in place. The air transmission pipe cost 55 cents per foot. If Mr. Young could see the pipe-laying that has been done by western engineers in the Sierra Nevadas, he would realize that there is very little of the earth's surface over which a pipe-line cannot be laid. His theoretical transmission line carrying 30 000 volts, in the present stage of electrical practice, is hardly worthy of consideration.

Mr. Young says that "electrically operated pumps and drills are fully as efficient as those driven by compressed air, and more portable." This, in regard to drills, is an unfortunate statement, as there has never been an instance of a drill successfully worked by electricity; electricians themselves admit this. In regard to electrical pumps, the best that can be said for them is that they have been a great trial and disappointment to miners.

The auxiliary set of boilers was put in as a safeguard against a possible break in the water supply, as well as in the air machinery. The duplication of electrical machinery would cost more than the boilers and would be of no use in case of a water stoppage; and the boilers, moreover, are used as air reservoirs.

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Mr. Foote.

The air at the works before reheating carries about 150 H. P., which is about 50% of the theoretical power of the water. The inconvenience of reheating is largely imaginary. In the case under consideration, the firing, which is done by the motor-man, amounts to handling a cord of wood in twenty-four hours; and the heated air is used at a distance of from 50 ft. to 400 ft., transmitted in magnesia-covered pipes which are not hot externally and give no trouble.

In answer to Mr. Young's incidental request for the figures on a 10 000-H. P. 20-mile air transmission plant, the author refers him for information in that direction to Prof. Unwin and Gen. Haupt. As a suggestion, however, the probabilities are that the air would be compressed to about 2 000 lbs., and could be transmitted in two 6-in. pipes, costing possibly $8 000 per mile, losing, say, 30 per cent. By the use at the terminal station of what Mr. Rix calls a pneumatic transformer, this 30% would be regained. This pneumatic transformer consists of a triple expansion Corliss engine reheating between the cylinders, exhausting at 90 lbs., and running compound air compressors pumping air into the distributing system at 90 lbs. pressure.

The first part of Mr. Doble's discussion, referring to comparative costs, etc., is answered in the foregoing reply to Mr. Young, and in the accompanying table of costs:

COST OF COMPRESSED AIR POWER PLANT AND PIPE LINE.

Power House:		
Grading	$895 86	
Foundation and building	6 437 42	
Machinery in place	7 410 00	
Total for power house		$14 743 28
Aqueduct:		
Grading	$50 65	
Masonry	1 384 35	
Total for aqueduct		1 435 00
Pipe Line:		
Trench	$6 156 10	
Masonry	600 17	
Pipe and receiver in place	20 199 95	
Total for pipe line		26 956 22
Air pipe transmission line		463 10
Reheater at works		560 00
		$44 157 60

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Mr. Foote.

Rights of way, legal expenses, engineering and superintendence are not included in the foregoing table.

In reference to changing the nozzle of the wheel, the author would state that this takes less than five minutes, and the air reservoirs (boilers) are sufficient to keep the works going while it is being done.

Theoretical discussions and comparisons have their interest, but the hard facts of practical working are what the engineer values most. The North Star plant, though employing well-known principles, is somewhat novel in its combination and application of them. Being the first complete plant of the kind, from start to finish, it was fully expected that mistakes would be made, and changes required causing delays and stoppages. However, on the first of February of this year, the sinking pump, weighing 7 tons, was swung into the shaft and lowered to the water level by an engine working with reheated compressed air. A few days later another pump was installed in another shaft, some 800 ft. away; air was led to it through a pipe, and reheated in an old hot water heater. The combined capacity of the two pumps was 1 250 galls. per minute, and for a few days over four months these pumps averaged a constant stream lifted from the old mine of over 1 000 galls. per minute. At no time was there a stoppage of an hour's duration in the air machinery. Since then and up to the present writing*, additional pumps have been put in, an underground engine installed, and the mines worked generally in the usual manner, and still without delay or stoppage.

The enormous experience that has been attained in the use of steam and steam machinery is very largely applicable in the use of air; men understand it and are not afraid of it. To this fact, the author believes, is due in great measure the record that has been made by this new plant. Another advantage in the use of the air system, which is of great importance in mining, is the ventilation so easily and cheaply supplied by the exhaust air. In the case under discussion, one pump throws air into the face of a drift 300 ft. away, another into a stope 200 ft. away, where 20 men are working, a third into the bottom of an incline shaft which is being sunk; and in every place, men go back to their work within ten minutes after blasting, and this is done without extra expense, except for the piping, which would be required in any system of ventilation.

* October 15th, 1896.

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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 789.

FLOW OF WATER IN WROUGHT AND CAST-IRON PIPES FROM 28 TO 42 INS. IN DIAMETER.

By ISAAC W. SMITH, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, 1896.

WITH DISCUSSION.

The water supply of the city of Portland, Ore., is from the Bull Run River, which heads in a lake fed by springs 3 500 ft. above sea level and 10 miles from Mount Hood in the Cascade Mountains. It runs with an average fall of 100 ft. to the mile over a bed of rocks and boulders between high walls of rock and hard pan into the Sandy River, a few miles above the place of its discharge into the Columbia. The water-shed of 220 square miles, now a government reservation, is mountainous, covered with timber, and unfit for settlement or occupation, and the main stream and its tributaries carry down but little sand or gravel, and not enough clay or alluvial soil to color the water during the highest floods. The temperature of the water, which averages about 38° Fahr. in winter, 45° in the spring and fall, and 55° in summer, does not change in passing through 30 miles of pipes.

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The minimum discharge is about 70 000 000 galls. in 24 hours, of which a third is now taken for the city water supply.

The water is diverted from the stream at the head of a rapid by means of a canal 400 ft. in length, terminating in a chamber from which it is discharged through a funnel 9 ft. in length and with a diameter tapering from 5 to 3½ ft., the latter being the diameter of the discharge pipe. A 42-in. gate is placed at a distance of 30 ft. from the head for the purpose of regulating or shutting off the water supply to the city. The head lost from the water chamber to a point below the gate is 6 ins. From the head to the foot of the canal there is a fall

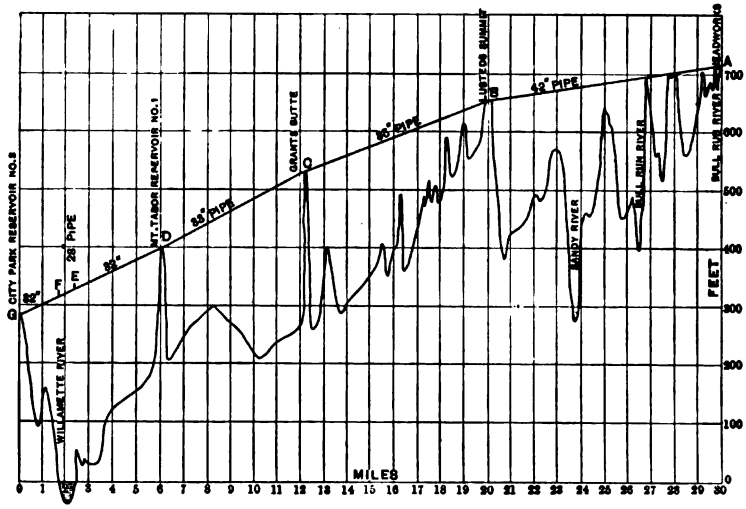

FIG. 1.

in the river of 16 ft. at low water, and as the canal walls are but 3 ft. in height, the flood waters are wasted over the top of the wall and there is but little variation in the level of the water in the water chamber.

The water is conducted to the city through a riveted steel and a cast-iron pipe, the one 24 miles and the other 6 miles in length. The steel pipe extends from the source to a reservoir on the east side of the Willamette River; the cast-iron pipe extends thence to a reservoir on the west side, passing under the river by means of a submerged pipe 2 006 ft. in length, as shown in Fig. 1.

At each reservoir the water ascends vertically into the gate chamber and flows through screens over a weir 6.97 ft. long with two end con-

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tractions, which differs from the regulation weir only in having a circular channel of approach. On the outside of the channel there is a a float supporting a gauge, which shows the height H of the water, and the corresponding discharge Q, calculated by the formula,

$$Q = 3.33 \text{ (length} - \frac{H}{5}) \times H^{\frac{3}{2}}$$

This float is adjusted by means of a hook gauge on the outside of the circular channel, but it is found that the height is 0.02 ft. greater on the inside.

The capacity of each reservoir was carefully measured, and the depth of the water in feet and the surface level for each 1 000 000 galls. were marked on a rod attached to the gate house and resting on the bottom.

On March 28th, 1895, 2 995 000 galls. by reservoir measurement was turned into Reservoir No. 1 in 3.7 hours, and 617 000 galls. by weir measurement into other reservoirs, making 3 612 000 galls. in 3.7 hours, or 23 410 000 galls. in 24 hours.

The flow by weir was 23 750 000 galls., or 340 000 galls. more than the reservoir and weir measurements combined. On January 21st, 1896, all of the water was turned out of Reservoir No. 1, and the full flow from Bull Run then turned in, and as the surface of the water rose to each million mark on the rod from 1 to 10, the time was noted and found to be 1 hour and 1 minute in seven instances, and 1 hour and 2 minutes in three, giving a flow of 10 000 000 galls. in 10 hours and 13 minutes, or 23 491 000 galls. in 24 hours. During the whole time the height of water above the weir stood steadily at 1.37 ft. on the outside of the circular approach and 1.39 ft. on the inside, giving a flow in 24 hours of 23 100 000 galls. for the first, and 23 592 000 galls. for the second, which differs only 100 000 galls. from the flow by reservoir measurement.

By tests made carefully at different times, it was ascertained that there was no appreciable leakage, either from the reservoir or from the supply main from Bull Run. As the weir at Reservoir No. 3 on the west side of the river was in all respects similar to that on the east side, it may be assumed that the weir measurements are correct in both cases, taking the heights on the inside of the circular approach.

The steel pipes are composed of 60-in. plates made up in alternate large and small sections, the smaller fitting at each end into the

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larger. The plates are double riveted on the straight seams, single riveted on the round, and coated with a preparation of asphalt, which rounds off the projecting edges of the joints and tends to lessen the frictional resistance on the pipes. The rivets were not countersunk.

On the 42-in. pipe, the thickness of the plates corresponds to Nos. 6 and 4, B. W. G., except for about ¼ mile on which it is ⅜ in. On the other pipes, 35-in. and 33-in., the thickness is 0.203 in., or No. 6, B. W. G. The diameters are estimated on the inside of the smaller courses, but during the progress of the work it was found that the pressure of the earth filling reduced the vertical and increased the horizontal diameters, the difference being from 1 to 4 ins., according to the depth of the fill and the care exercised in tamping under and around the pipes. To ascertain the effect of this distortion, sections of the pipe were subjected by means of jack screws and loads of earth to external pressures which reduced the vertical diameter within the limits of 1 and 8 ins.; but although without any load there was always a small difference in the transverse diameters, it was found that under any load the mean of the diameters was equal to the diameter of the circular pipe, and that the pipe when relieved from pressure resumed the original form and remained perfectly tight when subjected to a hydraulic pressure of 150 lbs. to the square inch. Assuming the section to be an ellipse the reduction of the area would, under the conditions stated, be inappreciable. Under a heavy water pressure from within, the pipes may have resumed the circular form, but on summits where the water pressure is light, the distortion still remains. The change of section is not apparent to the eye, but it is probable that it exists in all riveted or wrought pipes to an extent depending on the thickness and quality of the plates, the diameters of the pipes, and the depth and character of the filling.

Bends were made by means of joints not more than 6 ins. in width at the widest part, and riveted together in the same manner as the round seams of the pipes, each joint adding a round seam over and above the number required for a straight pipe.

Between the extremities *A* and *D* (Fig. 1) of the steel pipe there are two summits, *B* and *C*, which break the grade and necessitate pipes of different diameters: 42-in. on *A B*, 35-in. on *B C*, and 33-in. on *C D*.

The section *A B* extends from the head works *A* down Bull Run

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and across the Sandy River to the summit of the divide between that river and Kelly Creek. As the sides of the Bull Run Canyon are high and in some cases nearly vertical, the pipe line is necessarily thrown on high ground with many summits approaching nearly to the hydraulic grade line. From Sandy River to the summit *B* of the divide between that river and Kelly Creek, the rise is 400 ft. in 4 miles, with grades approaching in places to 45°, as shown on the profile. To avoid slides and vertical cliffs three bridges are necessary, two across Bull Run of 100-ft. and 200-ft. span, and a third of 300-ft. span across Sandy River. The ground is broken and exceptionally rough, requiring many bends, vertical and horizontal. The diameter of this pipe is 42 ins.

The next section extends from *B* along the north slope of the waters of Kelly Creek to the summit *C* of Grant's Butte, over broken and rough ground, requiring many bridges and trestles. The diameter of the pipe is 35 ins.

The next section, *C D*, extends from the top of Grant's Butte to the gate house of a reservoir known as No. 1, with no steep grades except the descent from Grant's Butte and the ascent to the reservoir.

From *D* to a reservoir, *G*, on the west side of the river, there is a 32-in. cast-iron pipe, and a section, *E F*, of 28-in. pipe passing under the river. The ground is comparatively level, but a considerable curvature is necessary in order to keep the pipe line within the limits of roads and streets.

The bends in the several pipes are as follows:

	Diameters of Pipes. Inches.				
	28	32	33	35	42
Total number of bends	2	17	85	141	225
Total degrees of curvature	21°	800°	404°	782°	1 913°
Average per mile	6°	14°	62°	101°	191°
Extra joints per mile for bends	....	....	11	29	42
Maximum radius	11'	11'	38'	38'	38'
Minimum radius	11'	11'	14'	38'	14'

The lengths of the pipes were calculated from the number of joints and checked by actual measurement. The elevations are measured by spirit level several times during the progress of the work and verified by a survey made recently.

For convenience of reference and calculation, the equations deduced

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from the hydraulic formula, $v = c\sqrt{rs}$, will be placed under the following forms:

v = velocity in feet per second.

d = diameter of the pipe in inches.

q = discharge in 1 000 000 galls. per day.

n = degree of roughness in the Kutter formula.

c = coefficient of discharge corresponding to n.

h = hydraulic grade, or loss of head per 1 000 ft. of pipe on the hydraulic grade line.

p = piezometer grade or loss of head per 1 000 ft. estimated on the piezometer grade line or lines; p cannot be greater than h, but may have any smaller value according to the discharge.

$$v = \frac{c\sqrt{d\,p}}{219.19} \quad \ldots\ldots(1)$$

$$q = \frac{c\sqrt{d^5\,p}}{62\,151} \quad \ldots\ldots(2)$$

$$c = q \times \frac{62\,151}{\sqrt{d^5\,p}} \quad \ldots\ldots(3)$$

$$p_{\frac{1}{4}} = q \times \frac{62\,151}{c\sqrt{d^5}} \quad \ldots\ldots(4)$$

$$o = l \times p \quad \ldots\ldots(5)$$

Equation (5) gives the loss of head in a length, l, of pipe.

To determine the discharge, q, through a series of compound pipes.

$$q = \sqrt{\frac{H}{o}} \quad \ldots\ldots(6)$$

In this equation H is the vertical distance or available head from the upper to the lower end of the compound pipe, and o is the sum of the losses of head on the several pipes calculated by equation (5) for a discharge of 1 000 000 galls. a day, or for $q = 1$.

TABLE No. 1.—DIAMETERS, HEIGHTS, AND HYDRAULIC GRADES FOR THE STEEL PIPES, *A B*, *B C*, *C D*, AND THE CAST-IRON PIPE, *D E*, *E F*, *F G*.

PIPES.				ELEVATIONS ABOVE CITY BASE.					
Material.	d.	Section.	l.	Point.	Height.	Point.	Height.	H.	h.
Steel.....	42″	*A B*	52 915	*A*	710.4	*B*	646.2	64.2	1.213
"	36″	*B C*	41 034	*B*	646.2	*C*	533.2	113.0	2.754
"	33″	*C D*	34 345	*C*	533.2	*D*	405.0	128.2	3.733
Cast iron..	32″	*D E*	21 460	*D*	401.8	*E*	—4.2	406.0	18.890
"	28″	*E F*	2 006	*E*	335.3	*F*	5.1	9.3	4.660
"	32″	*F G*	9 613	*F*	5.1	*G*	292.8	287.7	29.928

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The city base is 6.18 ft. above low tide at the low-water stage of the Willamette River. The elevations at the upper ends, *A* and *D*, of the steel and cast-iron pipes are corrected by deducting the head lost in generating the velocity. The elevation of *D*, 405, at the end of the steel pipe is the height of the water above the weir, but for the upper end of the cast-iron pipe it is the surface of the water below the weir. At the ends of the pipes the elevations are estimated to the tops of the pipes or to the surface of the water.

On February 27th, 1896, with the elevations of the water surfaces at *A*, *D* and *G*, as given in Table No. 1, the discharge was 23 500 000 galls. through the riveted pipes, and 20 750 000 galls. through the cast-iron pipe.

The 42-in. pipe did not run full beyond a point *A'* at an elevation of 648.5 ft., and 1 950 ft. east of the junction *B* of the 42-in. and 35-in. pipes.

The 35-in. pipe was partially full at the upper end, and at a point *B'*, 1 205 ft. west of *B*, the elevation of the hydraulic grade was 633.1 ft., by piezometer measurement. The elevation of the water surface was 532.5 at the junction of the 35-in. and 33-in. pipes, beyond which the 33-in. pipe was only partially full. At a point *C'* on the 33-in. pipe, 163 ft. west of *C*, the elevation of the hydraulic grade was 527.1 ft. by piezometer measurement.

On the compound cast-iron pipe, the elevations in Table No. 2 are those of the piezometer grades, calculated by equation (6). For the steel pipes the coefficients are calculated from the hydraulic grades.

TABLE No. 2.—DIAMETERS, GRADES AND COEFFICIENTS OF THE STEEL PIPES, *A A'*, *B' C*, *C' D*, AND OF THE COMPOUND CAST-IRON PIPE, *D E*, *E F*, *F G*.

PIPES.				ELEVATIONS ABOVE CITY BASE.							
Material.	*d.*	LENGTH, *l.*		Point.	Height.	Point.	Height.	*H.*	*h.*	*q.*	*c.*
		Section.	1 000 ft.								
Steel.....	42"	*A A'*	50.965	*A*	710.4	*A'*	648.5	61.9	1.215	23.5	115.9
"	42"	*A' B*	1.950	.....	.....	.....	.....	.....	.....	.....	.....
"	35"	*B B'*	1.205	.....	.....	.....	.....	.....	.....	.....	.....
"	35"	*B' C*	39.829	*B'*	633.1	*C*	532.5	100.6	2.526	23.5	126.8
"	33"	*C C'*	.169	.....	.....	.....	.....	.....	.....	.....	.....
"	33"	*C' D*	34.176	*C'*	527.1	*D*	405.0	122.1	3.572	23.5	123.2
Cast iron.	32"	*D E*	21.460	*D*	401.8	*E*	335.3	66.5	3.100	20.75	126.2
"	28"	*E F*	2.006	*E*	335.3	*F*	322.6	12.7	6.340	20.75	123.4
"	32"	*F G*	9.613	*F*	322.6	*G*	292.8	29.8	3.100	20.75	126.2

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Assuming p equal to h, and the lengths and grades to be as given in the preceding tables, the values of c and q for clean straight pipes, according to Hamilton Smith, M. Am. Soc. C. E., and J. T. Fanning, M. Am. Soc. C. E., and for $n = 0.0115$ in the Kutter formula, would be as follows:

TABLE No. 3.

d.	h.	Smith.		Fanning.		Kutter.			Table No. 2.
		c.	q.	c.	q.	n.	c.	q.	c.
42"	1.213	137.0	27.75	129	26.73	.0115	130	26.38	116
35"	2.754	136.5	26.41	125	24.19	.0115	124	24.00	127
33"	3.733	136.0	26.45	124	24.11	.0115	124	24.11	123
32"	3.100	135.0	22.15	123.5	20.26	.0115	123	20.19	126
28"	6.240	134.0	22.52	121	20.33	.0115	121	20.33	123

With the coefficients corresponding to $n = 0.0115$, the values of p, and the discharge of the compound 42-in., 35-in. and 33-in. pipes, calculated by equation (6), would be as follows:

TABLE No. 4.

d.	c.	Length.			Head lost $o = l \times p$.	Elevations of Grade.				q.
		Section.	1 000 ft.	p.		Point.	Height.	Point.	Height.	
42"	130	A B	52.915	1.048	55.2	A	710.4	B	655.2	24.5
35"	124	B C	41.034	2.871	117.9	B	655.2	C	537.4	24.5
35"	124	C D	34.345	3.852	132.3	C	537.4	D	405.0	24.5
		Total head lost.....			305.4	=	710.4 — 405.0			

The actual discharge, 23 500 000 galls., is 1 000 000 galls. more than was ordered, and 1 000 000 galls. less than would have been attained if the capacity of the 42-in. pipes had been proportional to that of the 35-in. or 33-in. pipes.

Of the three steel pipes the 33-in. was the straightest, and the coefficient should not have been 3% below that of the 35-in. pipe. The 42-in. pipe was over very rough ground, with many bends and summits, some of which rise very nearly to the line of the hydraulic grade, but the minimum radius of curvature was four times the diameter of the pipe, and there is no apparent reason for the reduc-

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tion of the coefficient and discharge to 10% less than it would have been on the supposition that the conditions were the same as those of the other riveted pipes.

When the water was first turned on, in December, 1894, the height of the water surface at the head works was about 710.4 ft., and the discharge by the floating gauge at the end of the 33-in. pipe was 24 900 000 galls., but as it has since remained steadily at 23 500 000 galls. under like conditions, either the gauge must have been out of adjustment or air must have accumulated at some of the numerous summits.

The air valves which have been placed on all summits have been examined; men have been sent through the whole length of the pipe to see that there were no obstructions, and that the 42-in. gate below the trumpet-shaped mouth-piece at the head was fully open; and the screens have been raised to see that they did not diminish the flow, but, although some of the flatter summits may have escaped observation, nothing has as yet been discovered which accounts for the deficiency in the capacity of the pipe.

It will be seen by reference to Table No. 3 that, except for the 42-in. pipe, the coefficients are about the same for the cast-iron and steel pipes, and agree nearly with the coefficients of Mr. Fanning for clean pipes and those of the Kutter formula, for $n = 0.0115$.

A. L. Adams, M. Am. Soc. C. E., engineer of the water-works recently constructed in Astoria, Ore., states that the supply main there is compounded of a 16-in. steel riveted pipe and an 18-in. wood-stave pipe; that the coefficient for the former corresponds almost exactly with the value given by Mr. Fanning for clean, straight cast-iron pipes, and to the value $n = 0.0111$ in the Kutter formula, and is only 84½% of the coefficient for the 18-in. stave pipe.*

The riveted pipes on which the experiments of Mr. Smith were made were put together stove-pipe fashion, and the high coefficients deduced may be due in part to the lesser frictional resistance resulting from this method of construction, but the projecting edges of the laps on the inside of a pipe are smoothed and rounded by the asphalt coating, and the increase of friction resulting from constructing the pipes in rings with large and small sections does not appear to be important when the thickness does not exceed ¼ in.

* See *Transactions*, Vol. xxxvi, p. 26.

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The assumption on which the preceding calculations are based, that the piezometer grade p, which would be the grade of a line along the water surfaces in a series of open-topped tubes extending upward from the pipe, coincides with the hydraulic grade h of a straight line from the upper to the lower end of a pipe is true only when the inner surface opposes a frictional resistance which is uniform throughout the whole length of the pipe. In such cases the coincidence of the grade lines depends solely on the uniformity and not on the amount of the frictional resistance. The discharge depends on the amount of the friction, and is proportioned to coefficients of discharge determined experimentally and corresponding to the roughness n in the Kutter formula.

According to the principles of the Kutter formula the coefficient c is independent of the slope in the equation $v = c\sqrt{rh}$, when r is greater than 0.1 ft. and h is greater than 1 ft. in 1 000 ft.

According to experiments on the flow of water through pipes the variation of the coefficient is greater, but the difference may be due, in part at least, to the use of the term in a sense not warranted by the conditions of flow in open channels.

In the case of an open channel with a constant discharge and with irregular cross-sections or a variable roughness of the sides and bottom, the velocity and volume of the water vary in accordance with the slopes and the resistances to the flow in different sections of the channel, and an obstruction placed in one section does not diminish the discharge or appreciably affect the velocity or volume of the flow in other sections, except for a short distance above or below the obstruction. In the case of a full pipe there can be no difference in the velocity or sectional area of the water, and an obstruction to the flow at any one point is followed by a change in the pressure against the inner surface at all points, which would affect the piezometer grades along the whole length of the pipe, and has the same effect on the discharge as a change in the hydraulic grade.

To calculate the flow of water in open channels by hydraulic formulas the hydraulic grade h must be used, and for pipes the piezometer grade p, which may be less, but cannot be greater than h.

A pipe laid on the hydraulic grade line $A\ B$ would run full with a discharge q, given by equation (2), and a grade h, and only partially full with a less discharge q'. Should the pipe be laid under the hy-

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draulic grade line, it would run full with the discharge q', but the grade would be reduced to p', calculated for the lesser discharge by equation (4).

Suppose two pipes with the same diameter and hydraulic grade, with discharges q and q' (the first calculated from the hydraulic grade), the reduction of the discharge from q to q' could be attributed to a greater roughness corresponding to a coefficient c', or to a change from the hydraulic grade h to the piezometer grade p', given by equation (4), and the values of c' and p' would be determined by the following equations, assuming the grades and coefficients to be independent quantities.

$$c' = c \times \frac{q'}{q} \text{ when } p' = h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

$$p' = h \times \left(\frac{q'}{q}\right)^2 \text{ when } c' = c \ldots\ldots\ldots\ldots\ldots\ldots\ldots (b)$$

The discharge would be the same in either case, but the pipe, if laid on the hydraulic grade, would run full with a roughness corre-

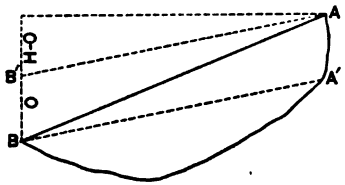

FIG. 2.

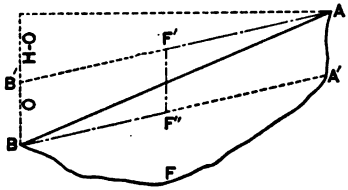

FIG. 3.

sponding to c' in equation (a), and partially full with the grade p' in equation (b), and a roughness corresponding to the coefficient c.

Should the discharge be decreased from q to q' through an open-topped pipe with a hydraulic grade line $A\,B$ (Fig. 2), the water would fall to a hydraulic grade line $A'\,B$, and the grade would be given by equation (4).

Suppose H to be the vertical fall from A to B, O the vertical fall from A to A', and l and k to be the lengths of the pipes $A\,B$ and $A\,A'$, the grade $h' = p'$ would be given by either one of the following equations or by equation (4):

$$p' = \frac{H - O}{l - k} = h \times \left(\frac{q'}{q}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots (c)$$

Suppose k to be small in comparison with l, the equation would be:

$$p' = \frac{H - O}{l} = h - o = h \times \left(\frac{q'}{q}\right)^2 \ldots\ldots\ldots (d)$$

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Should the supply be taken from a reservoir with a surface level fixed at A (Fig. 3), the hydraulic grade could not change, but the piezometer grade would be reduced from h to $h \times \left(\frac{q'}{q}\right)^2$.

Should the loss of head O be due to the partial closing of a stop valve F, causing a vertical fall in the piezometer line below F, the grades would be on the parallel lines $A'B$ or AB', should the valve be placed at A or B, and on the lines AF', $F''B$, should it be placed between A and B. In the latter case the grade lines would be on different sides of AB, but in each case O would be the vertical distance between the parallel grade lines; the grade p would be given by equation (d), and the pipe, if laid on the grade h, would be only partially full below the obstruction.

In these examples the whole loss of head has been supposed to take effect at one point, F, but should there be obstructions at several points, O would still represent the sum of the losses. The piezometer grades would descend in parallel lines between the obstructions like the steps of a stairway, and although there might be a considerable difference in the grades, the lines might continue close together, and would tend to coincide should the losses of head be equal and at uniform distances apart.

To the assumption that the piezometer grade is on the straight line between the heights of two points in the piezometer grade lines may be ascribed the variation in the coefficients deduced from piezometer measurements, so called.

For a series of compound pipes the loss of head for each one is determined by equation (5), and the discharge must be such that the sum of the losses in the several pipes must be equal to the total loss of head or vertical fall H from the upper to the lower end of the compound pipe, subject to the condition that the pipe must not rise at any point above the piezometer grade line. Examples are shown in Table No. 4 in the case of the 42-in., 35-in. and 33-in. pipes, and in Table No. 2 in the case of the compound 32-in. and 28-in. cast-iron pipe.

In a compound pipe the resistance to the flow of water in the several parts varies with the diameters, the degrees of roughness, and the local obstructions, and the piezometer grades fall in parallel lines on sections of uniform resistance, and vertically in the cases of local obstruc-

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tions, like the surface of the water of an open channel, but with the important distinction that while in one case the grades depend on the actual slopes in the different sections, in the other they are due to variations in the pressure from obstructions on the inner surface, the effect of each one of which is felt throughout the whole length of the pipe.

It follows that when the two end sections of a pipe present equal and uniform resistances to the flow, the piezometer grade lines must lie on different sides of the hydraulic grade line, *A B* (Fig. 4), and must cross it at one or more points, and that the coefficients calculated on the assumption of a straight grade line do not indicate the roughness of the inner surface.

The hydraulic grade line *A A' B* (Fig. 5), for instance, of the 42-in. riveted pipe falls at the rate of 1.213 ft. per 1 000 ft. to a summit *A'* west of the Sandy River, and the pipe is extended thence, along the hydraulic grade line, 2 000 ft. to a point *B*, at the end of the pipe.

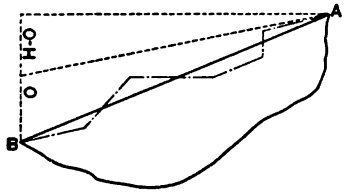

FIG. 4.

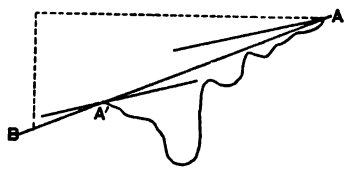

FIG. 5.

With a coefficient $c = 130$ (see Table No. 3), corresponding to the roughness of the 35-in. and 33-in. pipes, and to $n = 0.0115$, the discharge would have been 26 380 000 galls. in 24 hours, but it was only 23 500 000 galls. If the deficiency is due to the uniform roughness of the inner surface, the pipe would run full on the hydraulic grade line *A' B*, and the coefficient $c' = 130 \times \frac{23.50}{26.38} = 116$ would indicate a roughness equal to 0.0125. But the fact that the pipe does not run full from *A'* to *B* shows that the roughness is less than 0.0125 and that the grade is less than the hydraulic grade, and the reduction of the discharge may be ascribed to a reduction of the grade from $h = 1.213$ to $p' = h\,x\left(\frac{23.50}{26.38}\right)^2 = 0.963$, on account of bends, accumulations of air at summits, or causes not connected with the degree of roughness or the coefficient of discharge.

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A difference of 3 ins. in 1 000 ft. could not be detected readily, but measurements by pressure gauges show conclusively that the piezometer is above the hydraulic grade line at A and below it at A', and that the two lines cross at one or more intermediate points.

The difference of the grades multiplied into the length of the pipe, $o = (1.213 - 0.963) \times 52.9 = 13.225$, is the vertical distance between the parallel grade lines through A and A' and represents the total loss of head from irregular obstructions; but the difference in the elevations of the two grade lines, as measured by pressure gauges, does not at any point exceed 3 ft. Assuming, then, that there must always be a diminution of the grade from irregular obstructions, it will follow that the coefficients calculated by using the hydraulic grade in equation (3) are always too small, especially when o is small in comparison with h, and this may perhaps account for the great variation where the grades are light.

In Table No. 5 coefficients calculated from hydraulic grades are given from Mr. Fanning's tables and from the measurements of the 48-in. cast-iron pipes of the Boston Water-Works*; also the value of o in the equation, $p = h - o$, which would give a uniform value to the coefficients. The grades are in feet per thousand.

TABLE No. 5.

Authority.	Diameter, inches.	Grade. h.	c.	Grade. $p = h - o$.	c.	Pipes.
Fanning.....	6	1.409	96.3	$h - .020$	105	Clean straight.
"	6	2.976	101.7	$h - .032$	105	"
"	6	3.827	102.7	$h - .047$	105	"
FitzGerald.....	48	0.0115	116.5	$h - .003$	144	Clean.
"	48	0.0689	134.1	$h - .009$	144	"
"	48	0.3234	137.5	$h - .030$	144	"
"	48	0.7182	138.9	$h - .050$	144	"
"	48	1.2414	141.1	$h - .050$	144	"
"	48	1.8283	143.6	$h - .057$	144	"
"	48	0.0159	101.2	$h - .002$	110	Tuberculated.
"	48	0.0202	88.0	$h - .007$	110	"
"	48	0.5149	108.7	$h - .002$	110	"
"	48	1.1299	109.4	$h - .012$	110	"
"	48	2.6368	108.9	$h - .054$	110	"
Smith.....	42	1.213	116.0	$h - .250$	130	Steel riveted.

The coefficient 144 in the table is the same as that determined by Mr. FitzGerald for the 48-in. pipes with tubercles removed and a velocity of 6.195 ft. per second, but for the tuberculated pipes no definite conclusion can be drawn where the grades are very small, the

* See *Transactions*, Vol. xxxv, pp. 266, 267.

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coefficients being 114.8 in one case, for a slope of 0.0287 per 1 000, and 88 in another instance for a slope of 0.0211. This difference may be ascribed to the liability to error in measuring very small slopes, but for greater grades there does not seem to be so great a variation in the coefficients as with the clean pipes, which tends to support the principle of the Kutter formula that, except for very small slopes, the coefficients are constant for the same diameter of pipe, which would be more apparent should the slopes and coefficients be used as co-ordinates in diagrams, and the velocities be left out of consideration, or treated as functions of the grades. The effect of a small difference in the grades on the discharge will show the necessity of accurate measurements where the grades are light and the pipes not more than 1 000 or 2 000 ft. in length. For the 42-in. pipe, for instance, a difference of 3 ins. in 1 000 ft. makes a difference of nearly 3 000 000 galls. in 24 hours.

In regard to the effect of long use on the tuberculation of the inner surfaces of pipes, the following information is presented from experience with the pipe system of the city of Portland. The supply was first taken from small creeks, to which was added 28 years ago a plant for pumping from the Willamette River, which carries a large amount of mud and sand during floods. The use of this plant was afterward discontinued and a new pumping station erected 12 years ago, with a 4-mile force main of 30-in. riveted pipes of No. 6 wrought iron. The pump main laid 28 years ago was 18 ins. in diameter and of No. 6 wrought iron. It was laid on trestles across ground overflowed at high water, and was not protected from the weather. The coating was displaced on some portions of the pipe, but otherwise it appears perfectly clean and sound, and portions are now being used in the distribution system of the city under a pressure of 90 lbs. to the square inch. A portion of the 30-in. riveted pump main was taken up, and the coating was found to be intact and the pipe perfectly clean and sound. The cast-iron pipes from 6 to 24 ins. in diameter are free from tuberculation, so far as can be ascertained. The only exception is a 4-in. pipe which has been in use for 15 years and is rusted and rough on the inside, but it does not appear that this pipe was coated.

The Bull Run water is probably of the same nature as that of the Willamette River, except that it is always clear, and it is not probable that it will have a different effect on the pipes.

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CORRESPONDENCE.

Mr. Adams. ARTHUR L. ADAMS, M. Am. Soc. C. E.—In determining the coefficients for the 28-in. and 32-in. cast-iron pipes, the author seems to have treated the two as being similar in character, and, dealing with them as a compound pipe, has determined by calculation the relative losses of head in the two sizes. This treatment may have led to an erroneous conclusion regarding the carrying capacity of the 32-in. cast-iron pipe as compared with the steel riveted pipe. The 28-in. pipe is all of the flexible ball and socket joint type, a joint of this character occurring every 15 ft. 10 ins.* It is laid in the direction of flow, so that besides the necessary enlargement of section, the water impinges more or less against the end of the ball at every joint. Again, in laying a line of this character, it would seem to be unavoidable that there should be many angles, both vertical and horizontal. These do not seem to have been taken into account in tabulating the curvature. In view of these facts, especially the probable large resistance to flow afforded by the flexible joints, it would seem that had the loss in the 28-in. pipe been determined independently of that in the 32-in., in all probability the coefficient for the 32-in. pipe would have been found to be considerably greater than that stated. Such a result would accord more nearly with what might reasonably be expected, and would afford an explanation for what otherwise seems almost incredible in the paper as presented, that the cast-iron hub and spigot pipe possesses practically no advantage over the steel riveted pipe in discharging capacity.

Mr. Le Conte. L. J. LE CONTE, M. Am. Soc. C. E.—The lack of capacity in the upper section of the 42-in. pipe line, from the inlet down to *B*, may be naturally ascribed to two causes:

First.—The summits of the pipe line near the hydraulic grade line and the 2 000 ft. of pipe east of *B*, which is on the grade line, will always give rise to extra trouble, with constant accumulation of air near the summits. The fact that the capacity was 24 900 000 galls. per day, but soon after settled down to 23 500 000 galls. would seem to show a loss in capacity of 1 400 000 galls. or 5.6%, due to air obstructions alone. In many instances to the writer's knowledge these air cushions are seldom or never located right at the top of the summits, but at a certain distance from it toward the outlet, depending upon the velocity of the flow and the slope of the down-stream leg of the syphon. If the latter is a flat slope, the air cushion may be found some distance from the summit. The location of these troublesome air cushions is a special study in every pipe line and their position cannot always be

* See *Transactions*, Vol. xxxiii, p. 258.

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predicted with certainty, for the obvious reason that their location is liable to change, more or less, with every change in the velocity of the water column. The friction of the moving water against the bottoms of the air cushions causes them to take apparently abnormal positions along the pipe line. Consequently the selection of the best site to place a given air valve is more or less a tentative problem. Where the velocity of the water column is considerable, say 4 ft. per second, it sometimes happens that the first location of the air valve is not the proper one. These remarks apply with equal force when an engineer is called upon to fix the proper sites for his blow-off valves near the bottom of his syphons, to free his pipe line from sandy deposits. Mr. Le Conte.

Second.—The author states that in the middle portion of the upper syphon, *A B*, there is half a mile of pipe made of ¼-in. steel plates, No. 00 B. W. G. This heavy section is the controlling feature in fixing the value for the coefficient of roughness, *n*, in the Kutter formula. Recent observations by Emil Kuichling, M. Am. Soc. C. E., on the Rochester pipe lines show a marked variation in the value of the velocity coefficient, *c*, on the same pipe line where different sections were made of different thicknesses of metal. Experience in California also shows that on any given riveted pipe line the velocity coefficient, *c*, always diminishes with the thickness of the iron plates, which is perfectly natural. Hence it necessarily follows that in case of deep syphons the hydraulic grade is not a right line from the inlet to the outlet, but a broken line, or, more exactly speaking, a flattened reverse curve, having the steeper grade in the middle and flatter grades both above and below.

The author has stated that the 42-in. pipe, where built on the hydraulic grade, lacks about 4.5 ins. of flowing full. It is made of steel plates 0.203 in. thick. The actual discharge of this partly full section is found to be 23 500 000 galls. per day, and seems to establish a local value for *c* of 125 for this thickness of iron plate, which is reasonable. This being the case, it does not seem unreasonable to suppose that the heavier iron on both legs of the syphon, together with the half mile of ¼-in. iron pipe in the lower portion of the syphon, with sharp bends, would fully account for the balance of the loss in capacity, 4.5%, and thus make the effective coefficient of velocity, *c*, for the entire section, *A B*, approach somewhere near 116, which observation shows is the case.

It may be well to mention an extreme case in practice. The old Virginia City pipe line in Nevada, built in 1872, is a 12-in. wrought-iron riveted pipe in the form of a prodigious syphon, having a maximum pressure midway of 764 lbs. per square inch, due to a head of 1 720 ft. of water. The middle portion of the syphon called for plate iron $\frac{1}{2}\frac{1}{2}$ in. thick and correspondingly heavy rivets. The interior

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Mr. Le Conte. surface of the pipe was necessarily very rough. When the water was turned on, observations showed a low velocity coefficient, viz., $v = 78 \times \sqrt{r\,s}$.

Mr. Smith. ISAAC W. SMITH, M. Am. Soc. C. E.—Mr. Adams thinks that it is "almost incredible" that the cast-iron hub and spigot joint pipe should possess "practically no advantage over the steel riveted pipe in discharging capacity," and that if, for reasons stated, a lower coefficient had been assumed for the 28-in. ball and socket joint pipe, "the coefficient for the 32-in. pipe would have been found to be considerably greater than that stated." The 32-in. pipe, 31 073 ft. in length, was compounded with 2 006 ft. of a 28-in. ball and socket joint pipe under the Willamette River.

The 28-in. pipe was laid in a trench dredged to grade, on which there were two horizontal bends of 11 ft. radius, each of 21°, so that the curvature was not so great as on the 32-in. pipe.* The actual loss of head on the 28-in. pipe, measured as accurately as was possible by pressure gauges, was 11.2 ft., and the loss of head calculated in Table No. 2, for a coefficient of 123.4, was 12.7 ft., giving for the 32-in. pipe a coefficient of 126.2. As the lengths of the compounded 28-in. and 32-in. pipes were 2 000 and 31 000 ft., a reduction of 20% in the coefficient of one would not have resulted in an increase of more than 3% in the coefficient of the other.

Mr. Adams has stated† that he found the coefficient of a 16-in. steel riveted pipe on the Astoria Water-Works to correspond almost exactly with the value given by J. T. Fanning, M. Am. Soc. C. E., for clean, straight, cast-iron pipes of the same diameter. As the measurements have been accurately made, there is, therefore, nothing to discredit the conclusion that the coefficients are the same for the 33-in. steel riveted and the 32-in. cast-iron pipes.

The 33-in. and 35-in. pipes are all of No. 6 B. W. G. plates, 0.203 in. in thickness. On the 42-in. pipe there are 33 516 ft. of No. 6 (0.203 in.); 16 411 ft. of No. 4 (0.238 in.), and 2 703 ft. of ¼-in. thickness. The lower section of the 42-in. pipe, 2 000 ft. in length, is of No. 6 steel and laid on the hydraulic grade of 1.213 to 1 000. On this section the pipe does not run full, and the calculated coefficient of discharge is about 130, which corresponds nearly with the coefficients of the 35-in. and 33-in. pipes, taking the diameters into consideration. On the remainder of the pipe the coefficient calculated from the discharge is 116, which is about 10% less than 130. Of this, 5½% is ascribed by Mr. Le Conte to air cushions at or near the summits, and 4½% to additional friction from the increase in the size of the rivet heads in the No. 4 and ¼-in. plates.

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* See table of bends, page 201.

† See *Transactions*, Vol. xxxvi, p. 26.

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Mr. Smith.

In relation to the discharge of 24 900 000 galls., by weir measurement, for a few hours when the water was first turned on in December, 1894, the author is of the opinion that it was due to an erroneous adjustment of the gauge, because the water has since that date been repeatedly turned off and on, and the discharge has always been about 23 500 000 galls. During the past eighteen months a careful examination has been made to locate the possible air cushions near the summits. Additional air-cocks have been placed wherever there seemed to be a probability of the accumulation of air, and men have been twice sent through the whole length of the pipe, but in no instance has there been any evidence of deposits in the lower portions of the pipe or accumulations of air at or near the summits.

For these reasons, and because the many summits on the 35-in. and 33-in. pipes have not appreciably affected the flow, the author is of the opinion that the low coefficient of the 42-in. pipe, as compared with that of the 35-in. and 33-in. pipes, is not due to the accumulation of air at summits.

Between the sizes of the rivet heads for the No. 6 and No. 4 plates on the 42-in. pipe, there is no appreciable difference, and to reduce the coefficient from 130 to 116 on the 48 260 ft. of No. 6 and No. 4 pipes, would require a reduction of the coefficient for the 2 700 ft. of $\frac{3}{8}$-in. pipe to 20.4. For this reason, and because the rivet heads on the No. 6, 35-in. and 33-in. pipes do not materially reduce the discharge, the author ascribes the deficiency in the relative capacity of discharge of the 42-in. pipe, not so much to the larger size of the rivet heads on 27% of the length of the pipe, as to a misconception of the meaning and significance of the grade s in the hydraulic formula $v = c\sqrt{rs}$, and to the erroneous assumption that the pipe may be laid up to the line of the hydraulic grade without reducing the discharge.

Suppose, for instance, $A\ B$ (Fig. 2) to be the hydraulic grade (1.213 to 1 000) of a 42-in. pipe 50 965 ft. in length and with a roughness of the interior surface corresponding to a coefficient of discharge equal to 130; then, should there be no other resistance to the flow, the discharge in 24 hours would be 26 380 000 galls., as per Table No. 3, provided the pipe did not rise at any point above the line $A\ B$. Should the discharge be only 23 500 000 galls. the grade p, see (4), page 202, would be 0.963 per 1 000, and the loss of head due to other causes than the uniform roughness of the pipe would be 13.22 ft.

Should the water surface fall vertically a distance $O = 13.22$ from A to A' (Fig. 2) on account of an insufficient flow into the upper end of the pipe, the grade p and the hydraulic grade h would be on the line $A'\ B$ and equal to 0.963, and the discharge q, calculated by (2), page 202, would be 23 500 000 galls. a day, provided that no portion of the pipe should be laid above the line $A'\ B$ of the hydraulic and piezometer grade.

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Mr. Smith. Should the flow be reduced to 23 500 000 galls. by bends, gates or other local obstructions, the upper level of the water in the pipe remaining constant at A, the hydraulic grade h would remain constant at 1.213, but the grade p, calculated by (4), page 202, would be 0.963, and the piezometer grades would vary according to the nature and locations of the local obstructions.

Should the whole reduction of the discharge be due to the partial closing of a gate at the lower end of the pipe, $A\ B'$, parallel to $A'\ B$, and above the line $A\ B$ of the hydraulic grade, would be the piezometer grade. Should the gate be placed at F (Fig. 3) between A and B, $A\ F'$ and $F''\ B$, parallel and on different sides of $A\ B$, would be the lines of the piezometer grades.

Should the reduction of discharge be due to irregular obstructions at different points between A and B, the line of the piezometer grades would be broken, and although it might coincide nearly with that of the hydraulic line, there might be a considerable difference in the grades, and when determined on the assumption that it is on a straight line, the piezometer grade would not be the same as the true or hydraulic grade.

It is evident that the pipe should not be laid up to the line $A\ B$ of the hydraulic grade, when, as in Fig. 3, it is above the line $F''\ B$ of the piezometer grade, and it is questionable whether it should be laid up to the line $A\ B'$ (Fig. 2) or $A\ F'$ (Fig. 3) of the piezometer grade when it is above the line $A\ B$, or whether the pipe should be laid at any point above the line $A'\ B$ of the grade p, calculated from the discharge by (4), page 202.

On the 42-in. pipe the line of the hydraulic grade coincided nearly with the line of the piezometer grades determined by pressure gauges, and the pipe does not rise at any point above either line, but in the opinion of the author the deficiency in the discharge is due to the elevation of the pipe at three points within 2 miles of the head works to within from 4 to 7 ft. of the hydraulic grade line.

The coefficient c and the slope s in the hydraulic formula $v = c \times \sqrt{r\,s}$, have a definite meaning in application to an open channel, but not in application to a syphon pipe, and to this may be ascribed in part the variation in experimental results, but by making liberal allowances for the loss of head from irregular and local obstructions and giving a wide margin to the line of the hydraulic grade, the formula will be found to be "equalled by few and excelled by none."

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Vol. XXXVI. DECEMBER, 1896.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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No. 790.

THE NEW WATER-WORKS OF HAVANA, CUBA.

BY E. SHERMAN GOULD, M. Am. Soc. C. E.

PRESENTED SEPTEMBER 16TH, 1896.

WITH DISCUSSION.

The first attempt to supply the city of Havana with water was made in 1835, by the construction of the "Aqueducto Fernando Sétimo," which introduced the water of the Almendares River for public use. This supply was very inadequate. The water was diverted at a point about 4½ miles from the city, and, after passing through a rude and defective filter, was brought into the city in an 18-in. cast-iron pipe. This supply only amounted to about 1 333 000 galls. per 24 hours, and was liable to become very turbid from surface wash, the clarifying effect of the rude filtration being of slight account.

In order to obtain a better and more abundant supply, it was decided to collect in a suitable basin, a large number of springs which were found in the neighborhood of Vento, situated upon the River Almendares, about 10 miles from Havana. This idea seems to have originated with General Concha, and its execution was entrusted to

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General Francisco Albear, who at the time was President of the Board of Public Works of Cuba, and to whose genius and ability the credit of getting the project into practical shape is wholly due.

This new project was inaugurated November 28th, 1858, and in February, 1859, General Albear took charge of the work. The two following years were spent in surveys, examinations, collecting materials and other necessary preliminaries.

The springs, some 400 in number, giving a calculated yield of about 40 000 000 galls. per 24 hours, were collected in a large masonry-lined basin, with suitable overflows and sluices. These springs are situated near the river, on the farther bank from the city, and a high retaining wall forming one side of the basin prevents the entrance of the water of the river when it is swelled by freshets. A tunnel was constructed under the river, in which two lines of cast-iron pipe 1 m. in diameter were laid. These pipes connect with a masonry aqueduct about 6 miles long leading to the distributing reservoir at Palatino. This aqueduct is oval in section, about 8 ft. high and 6.5 ft. in maximum width, with a total sectional area of 41½ sq. ft., and an area below the spring line of the arch of 24½ sq. ft. The slope is $\frac{1}{5000}$, and the estimated velocity, when running to the level of the spring line, 2.43 ft. per second, delivering 59 cu. ft. per second, or about 38 000 000 galls. per 24 hours.

The first stone in the main wall of the collecting basin was laid June 26th, 1861. The first stone of the tunnel was laid in May, 1865. The water first ran through the pipes laid in the tunnel March 1st, 1872. The circumstances which brought about this slow progress are too numerous, complicated and uncertain to be entered into here. As it would evidently have been unwise to delay the delivery of at least a part of the water until the completion of the work, it was determined to connect that portion of the aqueduct already built with the old distribution. Accordingly, in June, 1872, the aqueduct was connected with the filter beds, already mentioned, and a partial supply of water, of improved quality, was thus obtained.

The total actual cost of the works executed up to the death of General Albear in October, 1887, according to his official statements, was under $3 500 000. These works included the receiving basin, tunnel and aqueduct. Of the thirty years which intervened between the commencement of the work in 1859 and its resumption in 1889,

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about to be described, there were only ten in which actual work was done, owing to want of funds and political disturbances.

Meanwhile the project of the new water supply had attracted the attention of American capitalists. The late Daniel Runkle, then President of the Warren Foundry and Machine Co., studied the project carefully, aided by the late J. C. Campbell, M. Am. Soc. C. E., then Chief Engineer of the Croton Aqueduct, who visited Cuba for the purpose, with the result that, in November, 1889, a contract for the completion of the works, comprising the building of the Palatino distributing reservoir and piping the city, was awarded to Messrs. Runkle, Smith & Co., of New York, who were represented in Havana by Mr. Richard Narganes, the president of the advisory board in Havana being the Marquis of Pinar del Rio.

The new works were inaugurated January 31st, 1890. The author, after a preliminary visit to Havana to report upon the project, was engaged by Messrs. Runkle, Smith & Co. as their engineer for the execution of this contract. He reached Havana in February, 1890, and preparations were at once commenced for work.

The contract for the entire work, including furnishing and laying the pipe, and building the reservoir, was taken at the engineer's estimate. The system of estimating public works in Cuba is somewhat peculiar. The plans having been prepared, an estimate is made of the exact quantities of each class of work required, down to the minutest detail. The plans, quantities and estimates, accompanied with a report, are then forwarded to the home government, in Spain, and if approved are forwarded back and can be acted upon. After such approval, it is extremely difficult to have any changes made, anything radical involving first acceptation by the proper authorities in Cuba, and then submission to the home authorities, and a royal order for the change. A feature which is frequently embarrassing is that the quantities and cost of each class of work must stand by themselves. If, in the execution of the work, the quantity, and consequently cost, of one class should fall short of that estimated, the surplus would not be available for making up the deficit in any other class which might overrun the estimate. Hence, the anomalous circumstance might occur, of being obliged to ask for a new appropriation for extra work, while there was still an unexpended and unexpendable balance on hand. The full set of documents of such a project comprise *planos*, *mediciones*, *presupu-*

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esto and *memoria*. The *planos* are the general drawings, illustrating the entire project, but only in a general way; the *mediciones*, or measurements and quantities, must be given, if expressible in cubic measurements, by the number of similar pieces of work, with their common length, breadth and thickness, which factors, multiplied together, give the total cubication. It will be readily perceived how inconvenient this rule is when dealing with pieces of masonry of irregular shape. In such cases the actual cubature must be first calculated, and then the amount divided up in such a way as to be expressible under the three dimensions of length, breadth and thickness.

There is another singular rule regarding the execution of public works in Cuba, which it is believed holds good in Spain also, which is, that implicit conformity to all plans and instructions given by the chief engineer does not relieve the contractor from the responsibility of failure, should it ensue, unless before commencing the work he files a written protest. In other words, his acceptance of a contract after examination of the plans and documents is held to be an approval of the design, which then virtually becomes his own, and for the success of which he, and not the engineer, is responsible. The want of reasonable foundation for this extraordinary regulation made it impossible for the author to believe in its existence until it was affirmed to him by unimpeachable authority.

The work to be done under the contract contemplated, besides the furnishing and laying of the pipe, the building of the distributing reservoir already mentioned. This reservoir is about 4 miles from the city. It is almost wholly in excavation, and consists of two compartments, or tanks, each containing about 8 000 000 galls. The bottom is covered with a concrete floor, and the sides are formed of rubble masonry retaining walls. From outside to outside of foundations of both tanks the area covered is 245 x 500 ft., or about 2.8 acres. The elevation of the bottom above city datum (which is understood to be mean low water) is 95.57 ft. The elevation of the lip of the overflows is 114.83 ft. The height of the retaining walls, from the level of the concrete floor to the top of the wall, is 20.5 ft. The top thickness of the wall is 2.79 ft.; its bottom thickness, not counting a small off-set, 6.73 ft. The face has a batter of 1 in 10, and the back is built in offsets. The area of cross-section is 97.20 sq. ft.

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Water is admitted to this reservoir from the aqueduct through an influent gate chamber. It may be admitted into either side of the reservoir, or both sides at once; or it may be shut off from either or both, and run through a masonry culvert in the center wall directly into the effluent gate chamber. The water may be also entirely shut off from the reservoir and gate chamber and turned into a waste culvert passing around the reservoir. Each side of the reservoir has an overflow discharging into the culvert, and there is another overflow in the aqueduct just before entering into the influent gate chamber. All the above operations are effected by sluice gates.

The effluent gate chamber contains a number of openings controlled by sluice gates, of which there are twelve in all used at the reservoir. From these openings the water enters a collecting pipe 42 ins. in diameter, running parallel to the side of the reservoir. Out of this pipe run the various other pipes destined for the distribution of the water to different parts of the city. These pipes, as well as the collector, are governed by valves. This part of the work will be referred to again. For the present it will suffice to say that the working of the system was perfectly satisfactory, although compared with the means usually employed in the United States to accomplish the same purpose, it appears unnecessarily complicated.

The original project contemplated a covered reservoir, which would seem to be especially proper in a tropical climate. The general drawings showed a series of granite columns, spaced 20 ft. apart from center to center, supporting a roof of concrete in the shape of groined elliptical arches, the concrete at the crown of the arches being about 1 ft. thick. The granite pillars were to consist, above the base block, of three stones each, respectively 7.38 ft. high by 2.63 ft. square, 6.56 ft. high by 2.30 ft. square, and 5.58 ft. high by 1.97 ft. square. These separate blocks were to be superposed, one upon the other, joined only by a bed of mortar, and were to be surmounted by a granite capital, forming a skewback whence the groined arches were to spring.

The great difficulties attending the execution of this design were apparent at first sight, and an endeavor was at once made to substitute a simpler and more common method for covering the reservoir. Several plans were submitted, with estimates showing their greater economy, safety, ease and rapidity of execution, the merit of which, as compared with the original design, was recognized by the highest

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authorities, both in Havana and Madrid, but they met with great opposition from the city officials, and, finally, after prolonged discussions, the question was left an open one as to whether there should be any cover built at all. There the question remains to this day.

The amount of money called for by the estimates was furnished by the Spanish Bank of the Island of Cuba, and was paid to the contractors in monthly installments as the work progressed, the estimates being signed by the engineer-director of the works, and the engineer-inspector appointed by the bank. The director of the works was Major Joaquin Ruiz, and the inspector, Major Ricardo Seco. Colonel Lino Sanchez acted as consulting engineer for Major Ruiz. All these gentlemen belonged to the Royal Spanish Engineer Corps.

The excavation for the reservoir was immediately commenced under the charge of Mr. Hector Simonetti. Disappointments occurred at the start from not getting the amount of Decauville plant of track and cars which had been ordered, but work was nevertheless started with all the means at command—plows, ox and mule carts, wheelbarrows, etc. The mule teams were soon discarded; each cart only took about 0.5 cu. yds., and, as it was made fast to the shafts, the harness had to be unbuckled to allow the shafts to be raised in order to dump. The oxen did somewhat better; the carts took 0.75 cu. yds. and could be more easily dumped, but the oxen were very slow and required much water and two hours' rest towards the middle of the day. Late in March, 1890, some wheel and drag scrapers were received from New York which the men could not at first use, and the wheel scrapers were never used satisfactorily. After the use of the drag scrapers had been acquired, they proved very efficient. By the last of April about 50 000 cu. yds. of excavation had been taken out from the reservoir site and the first section of the pipe line, which also required heavy cutting. Early in May some American dump cars and track were received, and a stationary engine erected, to haul the cars out of the reservoir. During the whole of the work, this engine was the main dependence in getting out the material.

Early in May, 1890, the author returned to the United States to collect plant, engage a force of American masons, and prepare for an active prosecution of the work as soon as the rainy season should be past. Mr. A. G. Midford was engaged as general superintendent of

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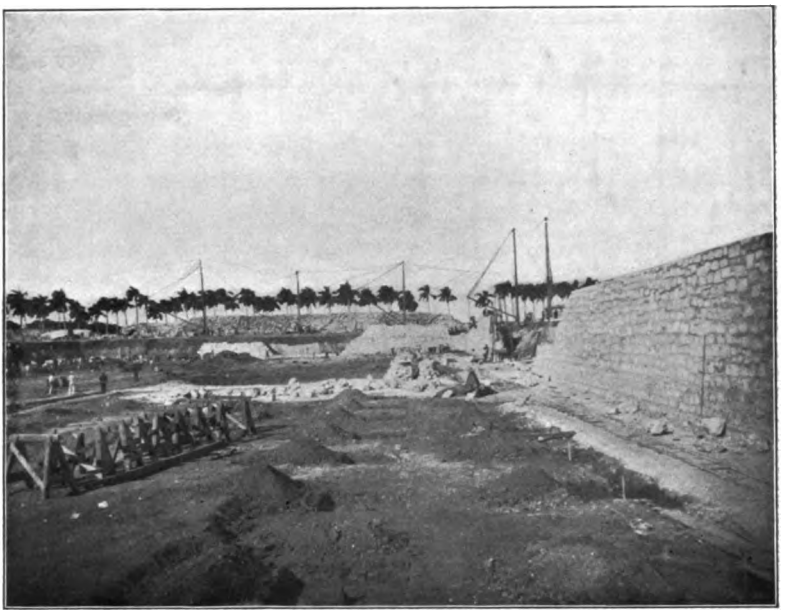

FIG. 1.

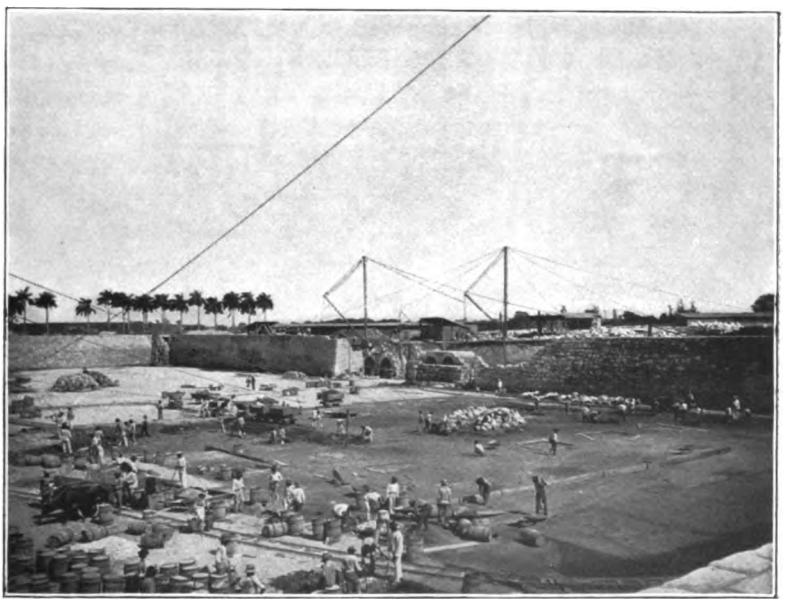

FIG. 2.

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The quantities and prices, in English measures and United States currency, of the pipe system, according to the final estimate, were as follows:

Cast-iron straight pipe, 7 930 tons at $85 39.....	$677 142 70
Lead, 320 682 lbs. at 6 cents.....	19 240 92
Hauling, 7 930 tons at $2 50.....	19 825 00
Pipe laying, 465 655 lin. ft. at $1.022 (average price)	475 899 41
Gates, valves, hydrants and various specials, 88.19 miles at $1 272.....	112 177 68
House connections and services, 88.19 miles at $760 10.....	67 033 22
Masonry, earthwork, pile-driving, etc., on 42-in. pipe line, 2.51 miles at $60 025 50.....	150 664 00
Unclassified	44 391 65
Total.....	$1 566 374 58

The manner of making up the engineer's estimate for this class of public works, according to Spanish rule, is to calculate as nearly as possible the actual cost at which it can be done, and then add 19% for contractor's profit. In the above statement, the 19% is added in for each item separately, as giving a clearer idea of the actual prices paid.

The length and weight of pipe, and the weight of the lead used for the different diameters, were as follows:

Diameters. Inches.	Length. Feet.	Weight. Tons (2 240 lbs.).	Lead. Pounds.
42.....	13 236	2 380	88 950
20.....	21 650	1 273	61 812
12.....	26 858	797	47 751
8.....	34 712	590	3 042
4.....	338 431	2 715	113 566
3.....	30 768	175	5 561
Totals.....	465 655	7 930	320 682

The construction of the Palatino Reservoir was under the author's exclusive charge, and he held, also, the position of consulting engineer for the entire enterprise covered by the contract of Messrs. Runkle,

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Smith & Co. The pipe laying was under the immediate charge of Mr. Simonetti.

Four pipes ran out from the collecting pipe in the effluent gate chamber. One 12-in. pipe was for the supply of the Cerro district, adjacent to the reservoir; one 20-in. for that of the Jesus del Monte district, also near by; another of 20 ins. diameter, to be connected, if necessary, with the old or Fernando Sétimo system, and the main 42-in. pipe for the general supply of the city. This last pipe extended about 2½ miles, crossing two valleys on masonry arcades, not included in the estimate for the reservoir, to an elevated point in the city, whence branches were run through the various streets.

The service pipes of wrought iron rapidly corroded in the impure soil in which they were laid, and were the cause of much trouble and expense for renewals.

The author desires to acknowledge his indebtedness to the gentlemen already named as connected with him in the work for the faithful, zealous and efficient co-operation which they rendered him in carrying out the undertaking. He is also indebted to Mr. Pelaez for the historical data given in the beginning of this paper.

Besides those whose names have been already mentioned, J. A. Ruiloba, Jun. Am. Soc. C. E., Mr. Fortun, a Cuban engineer, and Mr. Domingo Del Monte, were at different times employed on the work, and all did well in their several capacities.

The benefits accruing to the city of Havana by the execution of this work have been enormous. An abundant supply of exceptionally pure water has been introduced into all parts of the city, including those districts which previously were unprovided with any water except what was brought in pails from public plugs. It is true that, as the draught upon the supply increases, the pressure diminishes, and inconvenience has been already experienced from this cause. This inconvenience was apprehended and pointed out by the author when the work was commenced, and a diameter of 48 ins. recommended for the 13 236 ft. of main running out of the reservoir, instead of the 40 ins. originally contemplated. The extra cost was regarded, however, as prohibitive, and a diameter of 42 ins. was finally settled upon. The 48-in. main would in this distance have given over 15 ft. additional head at the point where the smaller mains branched off. That is, the calculated piezometric head at this point being at

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elevation 83.6 ft. with a 42-in. main, would have been at 99 ft. with one of 48 ins., a gain of about 18%, which, under the circumstances, would have been of immense benefit.

As a growing interest is taken in work in Spanish American countries, some general reference to this class of enterprise may be looked for in this paper. The experience and observation of the author in Cuba and elsewhere lead him to the following conclusions:

First.—The hope of reaping extravagant profits from such undertakings must not be entertained. No matter how favorable the contract or concession may be, a host of unforeseen difficulties are sure to arise owing to many causes, the partial enumeration of which, even, cannot be entered into here.

Second.—The work must be carried on with precisely the same economy, energy and attention to detail which would be considered essential to success at home.

Third.—As far as possible local help and materials should be employed, and methods of work made to conform to local usage.

Fourth.—No such enterprise should be undertaken unless sufficient capital has been secured to start and carry on the work rapidly and vigorously. The author is convinced that the striking success which, in spite of all obstacles, crowned the work just described, was very largely due to the sound and liberal basis upon which the undertaking was financed by its promoters.

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DISCUSSION.

Mr. Clarke. THOMAS CURTIS CLARKE, M. Am. Soc. C. E.—An interesting method of dealing with soft foundations has been followed by the engineers of the Canadian Pacific Railway in crossing muskegs or tamarack swamps. Gravel was formerly used, and an enormous quantity was required, as it would sink deep and spread out. Sawdust is now employed. About half the number of cubic yards, as compared with gravel, is needed to make a perfectly solid and reliable bank. This is covered with about 2 ft. of gravel and stands much better than the older work.

Mr. Wegmann. EDWARD WEGMANN, M. Am. Soc. C. E.—The sliding of the reservoir wall mentioned in the paper recalled to the speaker the movement of the abutments of a bridge on the New York, West Shore and Buffalo Railroad, about a mile north of Haverstraw, N. Y. At the place in question, an embankment 45 ft. high had to be made across a swamp to Cedar Pond Creek, which was to be crossed by a bridge of about 64-ft. span. As the swamp was very soft and about 20 ft. deep to hard bottom, it was decided to support the embankment by corduroy made of two courses of 6-in. saplings. This expedient was, however, of little value, as the weight of the embankment broke the corduroy and forced it to either side. The abutments of the bridge, which were about 45 ft. high, were constructed on solid pile foundations. During a freshet the creek, which had originally a depth of only 3 ft., scoured out its channel between the abutments so as to be 8 to 12 ft. deep at this place. Although the foot of the embankment approaching the south abutment was, at the time, at a distance of nearly 200 ft., the pressure in the swamp due to the weight of the embankment caused this abutment to move bodily about 1½ ft. towards the creek, without a crack being produced in the masonry. This movement was, of course, facilitated by the deepening of the creek.

The north abutment, except its wing walls, which were built on hard bottom, was also forced towards the creek by the north embankment, so that the span was reduced by about 2½ ft.

To stop the sliding, piles about 62 ft. long were bolted together in pairs, tip to butt, and placed as horizontal braces between the abutments, as a temporary expedient. In order to prevent any further scouring of the creek channel, sheet piles were driven across the creek just above and below the bridge. A crib-pier, reaching from one set of sheet piles to the other, was sunk half way between the abutments, stones being filled around the crib and between the sheet piling to bring the creek bottom to its original level. An invert of 12 x 12-in. timbers was then constructed so as to brace one abutment against the

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other, each of the timbers resting at one end on the crib pier and at the other against one of the abutments. This arrangement stopped all further movement, and has ever since held the masonry in place. Mr. Wegmann.

E. J. CHIBAS, Assoc. M. Am. Soc. C. E.—Some of the results of the Spanish method of awarding contracts in Cuba, referred to in the paper, are interesting. If the owner of a Cuban plantation desires to build a railway to some point on the sea coast or elsewhere, he has to file an application, before any work is done, with the Department of Public Works of the Province, even though the proposed work will be entirely on his own property. This application generally lies there from one to two years, according to the influence the planter can bring to bear on the Spanish officials. After it leaves the provincial office it goes to Havana, and finally to Madrid. After waiting from four to six years the petition may be granted, each stage in its progress requiring influence of one sort or another to push it along. The application then comes back to the island and the railway may be begun. In one case a small horse tramway was needed by a planter who was versed in the ways of the officials. He accordingly applied for a permit to build a regular railway; after some discussion a compromise was effected and he received his tramway permit, and saved a considerable part of the expense to which he would have been put had the original application been made out for the real object in view. It is on account of these laws, which make it necessary to spend several years and considerable money to secure permits, that many needed improvements have not been made in Cuba. The customs in the Spanish-American republics are not the same as in Cuba, and follow more the general practice in the United States. Mr. Chibas.

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CORRESPONDENCE.

Mr. Christian. G. L. CHRISTIAN, Assoc. M. Am. Soc. C. E.—The author has not mentioned some features of this work which it would be interesting to learn about, such as whether the excavation was in clay or sand, and whether the gates were closed for a definite time when the reservoir was first filled, and the subsidence, if any, noted, and the proportion due to evaporation estimated. It would seem as though a reservoir so well built and on which everything was done on such a liberal basis could not leak much, and the author has just cause for feeling proud of the strength displayed by the concrete, when only three months old.

It will be noticed that foreign Portland cement was used almost exclusively in the work mentioned, which reminds the writer of the time it was found necessary to remove a small part of the brick masonry in a reservoir gate-house to make a slight change from the original plans. The work had been done less than a year, and it was found impossible to separate the bricks, the whole being one homogeneous mass. The cement used in this work was an American Portland mixed in the proportion of two parts of sand to one of cement.

Mr. Low. EMILE LOW, M. Am. Soc. C. E.—It is stated in the paper that the Palatino Reservoir is almost wholly in excavation. The cubature of the extreme dimensions is about 90 000 cu. yds., while the amount of excavation in the final estimate is 89 474 cu. yds. There are also two more items of earthwork, 126 983 cu. yds. of embankment and 30 796 cu. yds. of terracing, concerning which more information is desirable. A considerable saving might apparently have been effected by reducing the depth of the cutting and utilizing the excavated material for an embankment back of the masonry wall. With a fixed width of embankment and a definite slope, the amounts of excavation and embankment could have been made to balance to a nicety. The nature of the soil or the position of the inlet and outlet pipes may have determined the location of the reservoir, but the paper fails to disclose the reason for the plan adopted.

The author deprecates the fact that large stone for rubble masonry was obtained with difficulty. Although this may have been detrimental to the contractor's pecuniary interests, it would seem that stone of medium size, bearing some ratio to the thickness of the wall, would be likely to make more water-tight work, on account of a greater possibility of breaking joints and the prevention of joints reaching through the wall from face to face. In this connection, attention is called to an accident which occurred at a masonry reservoir at Saltville, Va., used to store crude and clarified brine.

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Mr. Low.

The reservoir measures 195 x 165 ft. along the outside top edge of the walls. It is divided into four basins, two of which measure 120 x 75 x 7 ft., and two 72 x 57 x 12 ft. At the side of the reservoir there was a top layer of alluvial soil from 2 to 3 ft. thick, then a layer of clay of about the same thickness, and below this a bed of gravel from 4 to 6 ft. thick, resting on soft, friable limestone. The bottom was a bed of concrete about 1 ft. in thickness, made of an American Portland cement, sand and gravel mixed by hand. As soon as this bed became hard enough, the brickwork of the walls was started. It was proposed originally to lay the brick in cement mortar, but lime mortar was used, for economical reasons. The work was done by laying the concrete below the walls in trenches, starting the brickwork as soon as possible, and then excavating the earth in the basins and putting in the remainder of the concrete bottoms. These last were formed in sections by placing mold boards on the finished earth surface, parallel to the side walls at about 10-ft. intervals, and also along the diagonals of the bottom. As the work was done during a season when the temperature frequently dropped below the freezing point in the nights, fires were kept all night as near the fresh concrete as possible. Work was carried on during the winter whenever the weather was mild, and by spring the last bricks were laid.

With the advent of warmer weather, the inside walls were plastered with cement mortar. But little trouble was experienced with this plaster except on two sides of one of the deep basins, where surface and ground water was percolating through the wall. Portions of the surface of the concrete were badly disintegrated, and cracks were found following the lines of juncture of the sections in which the concrete was laid. It was then decided to plaster the bottom with cement mortar. Water was coming up through minute openings at irregular intervals in the concrete of the basin with leaking walls, and, after various expedients to get rid of it, short pieces of small wrought-iron pipe were driven into the openings, through which the water was allowed to issue. As it became warmer the plaster coat cracked and scaled off from the concrete, especially in the compartment giving the most trouble.

It was necessary to put the reservoir in service, but, after the basins were filled, brine was found to be percolating through the division walls, particularly those between the deep and shallow compartments. Tests showed that considerable fresh water was entering one of the deep compartments, reducing the strength of the brine and making it unavailable for the uses intended. The basin was pumped out, and water found to be issuing freely through the iron pipes. Large flakes of the plaster side lining were detached, and part of that on the bottom was also loose. These were removed and all defective surfaces replastered. Excavations were made for puddling the outside of the

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Mr. Low. walls of this particular division, when an accident occurred, threatening the safety of the entire reservoir.

About 40 ft. of the division wall (Fig. 1), between two of the basins, fell into the deeper compartment. At the time there was 5 ft. 10 ins. of brine in the shallow basin, and 6 ft. 6 ins. in the other, the difference between the respective water levels being 4 ft. 4 ins. The brine in the basins was drawn off as rapidly as possible, and it was found that the entire wall above the level of the shallow basin had been swept off, and the bricks thrown in a fan-shape mass nearly across the deep basin. The mortar was soft and did not adhere at all to the bricks. Several of the face courses of the lower portion of the wall were removed and replaced by bricks laid in cement and lime mortar. The upper part

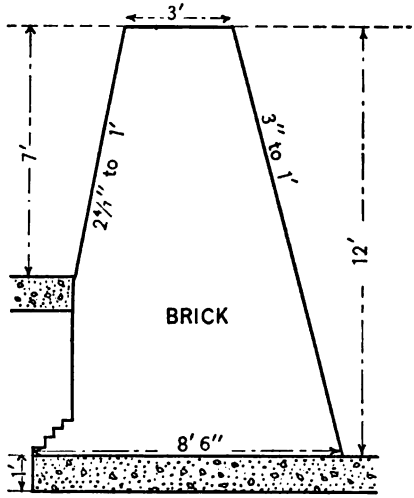

FIG. 1.

of the wall was relaid with the same mortar, using the old bricks as much as possible after cleaning them by a stream from a hose, which washed off the mortar.

The square corners at the connecting ends were rounded off, and the bricks on the faces of the lower parts of the walls of both deep basins were removed to a depth of 12 ins. and replaced by new bricks laid in cement mortar. Heavy counterforts, four in all, were built at the centers of the high walls and projected well into the basins. All loose plastering was replaced by fresh, and a prism of cement mortar placed along the bottom inside edges of all walls.

Mr. Le Conte. L. J. LE CONTE, M. Am. Soc. C. E.—Inasmuch as the practice of covering service reservoirs, located in tropical countries, is now gen-

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erally regarded to be the proper thing, it would be of interest to know what grounds, other than economy, the city officials had for objecting to the construction of a covered reservoir. Mr. Le Conte.

The author's remarks in regard to the relative value of machine and hand-mixed concrete naturally invite discussion. The class of mixer used, the proportioning being first done by hand, gives the best results, no doubt. Those who have had much experience with both kinds of mixings generally agree that the chief advantage obtained in machine-mixing is that of economy and not of excellence. The cost of labor is greatly reduced and the daily output is very much increased. These are great practical advantages which will always be the controlling factors in deciding the system to be adopted in most cases in every-day practice. On the contrary, wherever the concrete called for is of high grade and is to be subjected to heavy or violent shocks, the fact remains that hand-mixed concrete is always preferable, for the generally admitted reason that it is a more reliable mixture.

Among the weak points about machine mixing may be mentioned the fact that the plant is so cumbersome and inconvenient to move from place to place that it is rarely placed close to any particular piece of work, but at some distance, so as to be more central and more generally useful to the work as a whole. This necessitates transportation from the mixer to the tamping work, and here is where the trouble occurs. The writer has noted a large number of cases in which the freshly mixed concrete, after being transported, say 200 to 300 ft., and dumped at the site of a bridge pier, was found to be invariably more or less separated or unmixed, so much so that the final mixing was in point of fact actually done by the tampers in the bed-work. In case of hand-mixing, operations can always be held within circumscribed limits and thus the trouble about separation due to transportation may be largely avoided.

It is to be regretted that the author did not give more details as to original designs and modifications brought about by unexpected contingencies. Accidents will happen, and the facts and discussions which they bring out from experienced members are always interesting and instructive.

E. Sherman Gould, M. Am. Soc. C. E.—Mr. Clarke's remarks about the use of sawdust in railroad embankments are interesting and valuable, as are also Mr. Wegmann's on the method of shoring up bridge abutments, under certain peculiar circumstances. Mr. Chibas' observations on the peculiarities of official methods in the Island of Cuba are interesting supplements to those in the original paper. It may be further remarked that not only are the different classes of work kept distinct, as stated, but the work itself is divided up into parcels which are kept independent of each other. Thus, in the work described, there was a separate estimate for the reservoirs, the influ- Mr. Gould.

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Mr. Gould. ent and effluent gate chambers, the discharge culvert, etc., and there could be no giving and taking of quantities between these items, so that a shortage in one could not be made good from a surplus in another. Needless to say, that these cast-iron regulations necessitate the exercise of considerable ingenuity in making up the final estimate or *liquidacion*.

In answer to Mr. Christian, the material excavated was chiefly stiff clay. The completed work was turned over to the city, and it is believed that no such experiments as those suggested were made by the authorities.

In reply to Mr. Low, the reservoir was chiefly, though not wholly, in excavation, as the surface of the ground was somewhat irregular. The elevation of the bottom of the reservoir was fixed by that of the collecting basin at Vento, and the location was selected as being, upon the whole, the most favorable. A great deal of filling in, leveling, grading, etc., was done which is included in the item of embankment.

The facts, frankly given, regarding the collapse of some brine tanks, are instructive, as those relating to failures always are.

In answer to Mr. Le Conte's inquiry as to the motives of the city officials for their attitude in regard to the covering of the reservoir, the author can only reply, *Quien sabe?*

As to the relative merits of machine and hand-made concrete, the author would say that his preference is decidedly for the machine-made product, when the machine is properly operated. The labor of turning over a bed of concrete by hand, so as to thoroughly incorporate the ingredients is so great that it is apt to be slighted, and it is doubtful if the mixture is ever so complete as when the mass is churned up mechanically by steam power. In the present case, the product was fed directly into the wagons, which were then run down to where the concrete was to be used, and dumped bodily, either in place or in close proximity to it. It may be remarked that the Decauville, or other sheet-iron or steel, wagons are the best for this purpose.

Much might have been added to the paper in the way of details of the work, modifications of the original plans, etc., but the author's purpose was rather to give a brief and general statement of the leading features of an important foreign work undertaken and successfully completed by American capitalists.

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THE SUSPENSION OF SOLIDS IN FLOWING WATER.

BY ELON HUNTINGTON HOOKER, Ph. D.

PRESENTED SEPTEMBER 2D, 1896.

WITH DISCUSSION.

Considerable space in this paper is devoted to the historical side of the subject because the sources of information are widely scattered, and it is desired to indicate, so far as possible, the origin of the ideas and observations upon sedimentary movements which have become common knowledge. In the second part of the paper a comparison of particular facts and observations leads to certain general conclusions with reference to the manifestation of the phenomena studied. The concluding portion is devoted to an analysis of the different explanations of the cause of suspension, for the purpose of building up a satisfactory theory.

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To conduce to uniformity and clearness, the following symbols will be used throughout this discussion :

g = 32.2 ft. per second = acceleration of gravity.
F = area of right section of body considered.
h = head of water corresponding to the velocity v.
v = mean velocity of the stream in the vertical considered.
v_1 = surface velocity at the vertical considered.
v_0 = bottom velocity at the vertical considered.
i = inclination of the water surface.
f' = tangent of the angle of sliding friction.
f = tangent of the angle of rolling friction (properly a distance).
γ = heaviness of the liquid considered. For the purposes of this article, γ = 62.5 lbs. per cubic foot = heaviness of fresh water.
γ' = heaviness of the solid considered.
P = resultant thrust, in the stream direction, exerted by a moving liquid upon a solid.
V = volume.
G = weight.
M = mass.
k = constant determined by experiment.
Z = mean depth of stream.
z = variable depth below surface.
b = width of stream.
r = radius of sphere.
q = liquid discharge per unit width of stream.
d = solid discharge per unit of width.

Part I.—Historical Development of the Problem[1].

The varied phenomena incident to the flow of rivers have demanded the consideration and even the anxiety of riparian owners from a time far antedating the development of the modern science of hydraulics. Wherever rugged slopes discharge their melted snows or heavy rains are gathered from steep, impervious water-sheds, a mountain torrent has its birth, and the householder in the valley early learned to study its varying humors. The rapid descent of the Apennines to the sea

[1] It is proposed here to select, from the mass of literature touching upon this problem, only those discussions and observations which seem to mark a distinct ste toward its final scientific solution.

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and the consequent turbulent character of the streams of northern Italy, made this a fruitful field of study to a people whose scientific spirit had already gone far toward establishing the fundamental laws of fluid motion.

During the latter part of the seventeenth century means were sought for the amelioration of these mountain rivers. Dominique Guglielmini, physician and hydraulician, was employed by Venice and other Italian cities to prepare plans looking toward the prevention of their ravages. His greatest work was the building of the levees on the Po above Plaisance, and his writings[1] gave the first impetus to a scientific study of fluvial phenomena.

In 1773 Johann Silberschlag produced his comprehensive treatise on hydraulics[2], covering this field to some extent, but it was left to Dubuat[3] in 1786 to publish the first experimental studies which can be considered authoritative. These experiments were made at Paris by order of the French government. His determination of the different velocities at which solid particles begin to be moved by flowing water has been accepted by subsequent writers, and with his work begins the scientific knowledge of the movement of alluvions in sedimentary rivers.

Dubuat's artificial canal was formed of planks 12 pieds[4] in length, 3 pouces thick and 18 pouces wide, so fitted that the form could be altered from rectangular to trapezoidal by the addition of supplementary bracing. Its total length was 132 pieds, and in the trapezoidal form it had a clear bottom width of 5¾ pouces to a surface width of 3 pieds. Water was discharged into the canal under a maximum head of 7 to 8 pieds.

Surface velocities were measured with floats, noting the time of traversing 10 toises[4]. For determining bottom velocities a small ball of mastic was first adopted. Its specific gravity was such that it lost ¼¾ of its weight in water, and was thus very easily moved. Later, more satisfactory results were obtained with red currants. These were

[1] "Della natura di fiumi trattato fisico matematico." Guglielmini, Bologna, 1697.

[2] "Hydrotechnik oder des Wasserbaues." Johann Silberschlag, Leipzig, 1773. Copy at Zurich Polytechnikum.

[3] "Traité d'Hydraulique." Dubuat, Paris, 1786. Page 57, Volume I. Third edition published in 1816. Copy at Zurich Polytechnikum and at École des Ponts et Chaussées, Paris.

[4] "Système ancien" of France—

1 pouce = 1.066 ins.
1 pied = 1.066 ft.
1 toise = 6.395 ft.

These units were in use previous to 1812. Between 1812 and 1840 the "système usuel" was in vogue. Its values are slightly larger—1 pouce = 1.093 ins.

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smoother, moving with less friction, and could be more easily seen. The time of passing through a distance of 60 pieds was noted.

The bottom velocities at which various materials began to be moved by the current were as follows:

Potter's clay (beginning with a velocity of 45 pouces, it continued to be carried away as the velocity was gradually decreased to 7 pouces. At 7 pouces a deposit of fine sand took place, which continued on down through a velocity of 4 pouces, until at 3 pouces per second the clay ceased to show action)	3 pouces per second.
Gravel (size of anise seed)	4 pouces.
Gravel (size of peas)	7 "
Coarse sand (sand remained stable while bottom velocity was increased from 3 up to 7 pouces. At 8 pouces it began to be entrained and for velocities of 12 to 45 pouces per second it continued to be entrained and suspended)	8 "
Sea pebbles (1 pouce diameter)	24 "

Dubuat's experiments also showed that a current velocity of 10 or 12 pouces per second was sufficient to produce sand waves in a bottom whose grains were large enough to be easily visible. He describes these furrows as perpendicular to the longitudinal axis of the current with a short steep down-stream face and a long gentle posterior slope. Each sand grain was slowly rolled along the up-stream incline and fell of its own weight down the crest, thus advancing the wave by steps equal to the diameter of the grains.[1] He computes a velocity under these circumstances which requires two years to cover a length of 2 400 toises.

The expression now universally used to represent the thrust exerted by a current against a solid of any form was deduced by Dubuat[2] and the coefficients experimentally determined.

He argued that the pressure on the up-stream face (p_1) would be greater and that on the down-stream face (p_2) would be less than the

[1] For a similar description see "Report of Chief of Engineers, U. S. A.," 1875, II, pp. 502–504.

Also "Handbuch der Wasserbaukunst." G. Hagen, 1871, Zweiter Theil, "Die Ströme," S. 161–162.

[2] See Flamant, "Hydraulique," 1891, p. 561.

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pressure (p) at the corresponding points if the solid were removed. Therefore, the total impelling force would be

$$P = F(p_1 - p_2) = F[(p_1 - p) + (p - p_2)].$$

These pressures can be written as functions of the velocity height h, whence

$$\frac{p_1 - p}{\gamma} = m\frac{v^2}{2g} \text{ and } \frac{p - p_2}{\gamma} = n\frac{v^2}{2g},$$

when m and n are experimentally determined.

By substitution,

$$P = (m + n)\,\gamma\, F\frac{v^2}{2g} \text{ or, as usually written,}$$

$$P = k\,\gamma\, F\frac{v^2}{2g}.$$

The fact that floating solids move with a velocity superior to that of the current which bears them was noted by Dubuat[1]. His explanation of it was inaccurate, but the phenomenon itself has an important bearing on the present discussion.

Of interest in this connection is the experimenter's statement with reference to the theoretical form of bed best adopted for flowing streams. He rejects the rectangle and semicircle as being unable to sustain their own weight in soft soils and chooses the trapezoidal cross-section as offering a proper talus. These right lines will be rounded by the stream itself, the slope being proportioned to the diminishing velocity from center to sides.[2]

Dubuat's work further includes various studies into the régime of rivers and the development of rules for that radius of curvature at bends which will best conduce to stability. To him belongs the honor of inception along these lines.

J. A. Fabre was the next to publish systematic studies[3] on the movements of solids in torrents and rivers, followed in 1811 by the voluminous encyclopedia of Wiebeking.[4] These men extended the range of observed data without making material additions to the theory of fluvial action.

In the year 1845, Bouniceau[5] discussed at considerable length the

[1] "Principes d'Hydraulique," Dubuat, No. 220. Quoted by Durand-Claye, *Annales des Ponts et Chaussé-s*, 1886, 1, 530.

[2] See "Principes d'Hydraulique," Dubuat, 1786, Vol. I, p. 119.

[3] "Essai sur la théorie des torrents et des rivières," J. A. Fabre, Paris, 1797, Première Partie.

[4] "Wasserbau." C. F. Wiebeking, Munich, 1811–1817, 4 volumes.

[5] "Étude sur la navigation des rivières à marées et la conquête de lois et relais de leur embouchure." Bouniceau, Paris, 1845.

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shifting of sands in tidal estuaries, and showed himself a close student of the laws of erosion by water action. His excellent little volume brings to light and discusses a number of anomalies in this form of action.

He gives a set of values of the bottom velocities at which erosion begins to take place with different materials :[1]

Clay	0.08 — 0.15	m. per second.
Coarse sand	0.22 — 0.30	"
Coarse gravel	0.11 — 0.61	"
Ordinary pebbles	0.65 — 1.00	"
Stones (size of an egg)	1.00 — 1.20	"
Conglomerates	1.52	"
Sedimentary rock	1.83	"
Solid rock	3.00	"

The River Garonne for a length of 45 miles below the embouchure of the Lot was made the object of a series of observations covering 11 years by M. Baumgarten.[2] These measurements deal with the varying discharges from month to month, with the constant changes in form of cross-section and maximum depths, determinations of the fall and heights of water as well as with geological and meteorological studies of the valley. It has formed the model for later fluviatile studies. Here are given the first measurements of discharge of detritus[3] which the author has been able to find. Daily samples were taken at Marmande from the surface of the river in a vessel containing 4.6 liters. This was allowed to stand for nine or ten days, the clear water decanted and the sediment filtered until thoroughly dry. After weighing, a simple calculation gave the weight in grams of mud per cubic meter of water. These measurements were continued from 1839 to 1846 continuously, and the average monthly solid discharge of the river at this point computed.[4] M. Baumgarten distinguishes three different methods of movement common to these solids.

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1 "Etude sur la Navigation," etc., p. 19.

2 "Navigation fluviale, Garonne." M. Baumgarten, Ingénieur ordinaire. *Annales des Ponts et Chaussées*, 1848, 2, 1-157.

3 Same, pp. 47, 146.

4 In order to see if the water contained the same amount of suspended matter at all depths, Baumgarten made a series of tests of specimens from different depths and taken from points where the velocity was different.

From the results given in the table in the continuation of this note on the opposite page he decided that a surface specimen gave a fair average.

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First.—A discontinuous rolling motion along the bed of the stream which takes place when the velocity of the current is limited or the materials large.

Second.—With greater velocities or smaller particles, a discontinuous suspension in the lower laminæ of the current.

Third.—Movement in continuous suspension when the particles are carried throughout the entire length considered.

The sand waves which M. Dubuat had observed on a small scale are reproduced in the Garonne on a large scale in gravel shoals, and M. Baumgarten made careful measurements of the yearly progress of one of these crests.

In 1840 the talus down stream had a vertical height of 1.3 m., a base of 2.8 m., while the length of the crest was 180 m. In 1841, the form was nearly the same, but the crest had moved down stream parallel to itself about 30 m. In 1842 the forward motion was 20 m. These gravels were of about the size of a walnut, and the velocity of the water averaged 2.25 m. per second.

Thus far attention had been especially directed towards the phenomenon of dragging, and the laws it follows had been, to some extent, investigated. Inspecteur-Général Dupuit,[1] in 1848, emphasized the true importance of suspension in the movements of soft river bottoms, and to him is due the first scientific study of the causes which produce this action.

Dates.	Depth at which the water was taken.	Weight of Filtered Sediment.	
		In dead water of a bridge or in a gentle current.	In a strong current.
		Grams.	Grams.
March 25th, 1847..	at the bottom at 7.0 m....	0.72	....
	at 3.5 m.....	0.75	....
	at the surface.....	0.82	....
March 27th, 1847..	at the bottom at 8.0 m....	0.34	....
	at 4.0 m.....	0.32	....
	at the surface	0.19	....
April 9th, 1847....	at the bottom at 8.75 m...	1.29	0.93
	at 4.40 m.....	1.43	1.18
	at the surface.....	1.13	1.22
April 15th, 1847....	at the bottom at 9.0 m....	1.89	1.90
	at 4.50 m.....	1.78	2.15
	at the surface.....	....	1.60
April 18th, 1847....	at the bottom at 8.0 m....	0.94	0.90
	at 4.00 m.....	0.87	0.68
	at the surface.....	1.33	0.87

[1] "Etudes théoriques et pratiques sur le mouvement des eaux." Paris, 1848. Second edition, Paris, 1863, pp. 214–229. J. Dupuit, Inspecteur-Général.

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Dupuit calls attention to the experiment of revolving rapidly a glass of water containing sand grains. He notes that there is a direct relation between the velocity of the water and the amount of sand in suspension, and that the grains tend to arrange themselves in successive laminæ according to the order of their size; as the velocity is decreased, they descend successively to the lower strata. These facts had all been observed before his time, but Dupuit goes farther than his predecessors in noting that the maximum amount of suspension, *i. e.*, that in the lower layers, corresponds, not to the greatest absolute velocity of the current, but to the maximum relative velocity of contiguous molecules. This is a distinct step in advance. Dupuit finds here his explanation of the phenomenon of suspension.

Starting with the fact first noted by Dubuat that the velocity of a float exceeds that of the current,[1] he calls attention to the tendency of such bodies to move toward the filaments of greatest velocity and explains this upon the principle of least work. Assuming the resistance to its motion to vary with the direction of its path, this direction will necessarily be that which offers the least resistance. Therefore an oblique path toward the most rapid current in the stream line will result, since this will offer the least difference in velocity between the solid and the fluid and so the least frictional work.

Dupuit derives a law for this lateral movement as follows:

Let v = the absolute longitudinal velocity of the body.

u = the absolute transverse velocity of the body.

w = absolute velocity of filament at shore side of body.

w' = absolute velocity of neighboring filament toward center of stream.

The relative velocity of the body as regards the liquid surrounding it may be expressed by:

$$\sqrt{u^2 + v^2} - \frac{w + w'}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Considering the resultant of the resistances which the body suffers as approximately proportional to this relative velocity, the value of u may be found for which this resultant is a minimum.

Calling Q the angle of inclination of the tangent to the curve of velocities at the point considered,

$$w' = w + u \tan. Q \ldots\ldots \ldots\ldots\ldots\ldots\ldots (2)$$

[1] Dupuit, in common with Dubuat, ascribes this excess of velocity to the accelerating force represented by the component of the bodies' weight parallel to the surface of the current. M. Du Boys, *Annales des Ponts et Chaussées*, 1886, 1, 199-242, has clearly demonstrated the incorrectness of this explanation.

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Substituting (2) in (1) and putting the first differential coefficient of the expression equal to zero, he finds that the resistance will be a minimum for

$$u = \frac{\tan. \frac{Q}{2}}{2} \sqrt{u^2 + v^2}$$

i. e., the transverse velocity should decrease with the tangent of the curve of surface velocities, or, in other words, from the banks to the center of the stream. This is equivalent to saying that the maximum lateral velocity will correspond to the maximum relative velocity of the filaments.[1]

Applying this same law to the velocities considered in a longitudinal section, he finds a resultant force acting obliquely upward, which produces the phenomenon of suspension. As this force will be greatest where the relative velocities are greatest, *i. e.*, near the bottom, the lower laminæ will carry the heavier load of particles. Solids of equal density will arrange themselves from bed to surface in the order of their volume. Suppose, now, the relation of a solid to neighboring ones is considered. The presence of another will tend to decrease the relative velocities of the filaments, and so the two will be obliged to descend to a lower lamina than would the one alone. Descent or ascent will follow according as the bodies approach each other or separate.

Dupuit formulates these laws as follows:

"*First.*—A water current can suspend solids of a density superior to its own.

"*Second.*—The power of suspension depends upon the relative velocity of the filaments and is greater according as this relative velocity is greater. In general, it is proportional to the quantity $\frac{dv}{dz}$ (where v = velocity of current and z = depth below surface) so that lower layers can carry either more solids or those of greater volume.

"*Third.*—The power of suspension of a bed is limited, *i. e.*, a square meter of cross-section can only carry a certain number of solids of a definite volume. Thus each lamina has a different degree of saturation."

[1] Observations made by Major Cunningham in the Ganges Canal seemed to indicate a current from the shore to mid-stream whose intensity followed this same law (see *Proceedings* of the Institution of Civil Engineers, Vol. LXXI, p. 66). As this current was indicated only by the behavior of certain floats, it is more in consonance with present knowledge to believe their action due to the cause given here by Dupuit than to suppose an actual lateral motion of the water.

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Dupuit assumes a river flowing with section and fall unchanged, and saturated with sediment. The entire load will be carried to the embouchure. Suppose the section to vary. At each change will come a change in the curve of velocities and a consequent change in the power of suspension.

When this power is reduced, there will follow a deposit, and when it is increased, erosion will take place. He makes it clear also that these results are dependent, not only upon the section at the point where the change takes place, but also upon the anterior portion of the river as affecting the state of saturation in which the river reaches the section in question. These effects can be brought about at any point whatever by suitably changing the up-stream section.

When a deposit occurs the material comes wholly from the lower laminæ, and they, in turn, receive from the upper ones the material in excess of their power of suspension. This explains the lamination of river-beds in materials increasing in size with depth below the bottom. The frequent presence of beds of finer particles interrupting this structure he explains by the principle that saturation may be obtained either by the size of the particles or by their nearness together.

The numerous variations to which this laminated movement is subject is noticed, and explains the constant rising and falling of particles from one lamina to another, while the nature of the horizontal curve of velocities is such as to cause a constant movement of particles from the banks toward the center. To this may be attributed the tendency of a river to form islands in the middle of its bed at the expense of its banks.

Among the German writers of this period, the discussion of the transportation of stones by torrents was especially taken up by Joseph von Gumppenberg Pöttmes[1], but no further experiments were published until 1857, when Blackwell[2], in England, extended the investigations of Dubuat to solids of larger dimensions.

The velocities given in the table on the next page are those at which movement began:

[1] "Der Wasserbau an Gebirgsflüssen," Joseph Freiherrn von Gumppenberg Pöttmes, Augsburg, 1854.

[2] See "Report of the Referees upon the Main Drainage of the Metropolis," July 31st, 1857, Appendix IV. Also, for table here quoted, see *Proceedings* of the Institution of Civil Engineers, Vol. 32, p. 48.

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Description of substance.	Cubic Contents.		Velocities.		Increase of contents of substances moved.	Increase of velocities.	Sixth root of increase of contents.
	1. Cubic inches.	2. Cubic inches.	1. Feet per second.	2. Feet per second.			
Brickbats.....	2.59	18.5	1.75 to 2.00	2.75 to 3.00	7.14	1.37 to 1.70	1.38
Brickbats.....	4.76	18.5	2.25 to 2.50	2.75 to 3.00	3.97	1.10 to 1.33	1.26
Oolites..... ...	2.39	17.68	2.00 to 2.25	2.75 to 3.00	7.40	1.22 to 1.50	1.39
Flints.....	1.95	10.37	2.50 to 2.75	3.00 to 3.25	5.32	1.09 to 1.30	1.32
Slate:.....	2.38	9.06	2.00 to 2 25	2.75 to 3.00	3.81	1.22 to 1.50	1.25

The important idea of saturation with solid material is definitely stated[1] by M. Scipion Gras in a valuable paper[2] published at this time.

He defines saturation in a stream as that state at which the least addition to the solid material already carried will cause a deposit, and its power of entrainment as the total weight of material which a given stream in a state of saturation can carry. He assumes this power of transport to vary directly with the velocity, density and depth of the water, and, these quantities remaining constant, to vary with the volume, density and form of the solids submitted to its action. Upon these principles he explains erosion as a necessary consequence, when the saturation corresponding to the actual velocity is incomplete, unless the bed offers too great a resistance.

Measurements of the advance of the crests of shoals, similar to those undertaken by Baumgarten in 1840, were made by Hübbe[3] in 1861 on sand bars. His observations show the same wave form on a large scale, which Dubuat had noticed in the minute form, and confirm Baumgarten's statement of the forward motion of the crests.

The results of the exhaustive study of the Mississippi River[4] and

[1] Probably first stated by Frisi, "On Rivers and Torrents," 1732. See Report on Mississippi River. Humphreys & Abbot, pp. 190, 415.

[2] "Études sur le torrents des Alpes." M. Scipion Gras, *Annales des Ponts et Chaussées*, 1857, 2, pp. 1-96.

[3] *Zeitschrift für Bauwesen*, Jahrgang xi, 1861. Abstracted in *Zeitschrift des Architekten und Ingenieur-Vereins*, Hanover, 1863, p. 518.

[4] "Report on the Mississippi River." Humphreys and Abbot, 1861. Reprinted with additions, Washington, 1876.

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its delta were published in 1861, and contain a wide range of data on the distribution of sediment. Observations along the same lines as those of Baumgarten were instituted at Carrollton in 1851, and lasted throughout two years. They were conducted by Prof. Forshey. Samples were taken from a point near the east bank, where the high-water depth was 100 ft., from the middle of the river, and from a point near the west bank, where the high-water depths were 100 and 40 ft., respectively. These tests were made daily (except Sundays) and samples taken from surface, mid-depth and bottom by means of a small weighted keg, with valves opening upward, which was designed to allow free passage to the water until it reached the desired depth. At the station near the west bank only surface and bottom samples were taken. An average value was obtained for the weight in grams of sediment to 600 grs. of water at each of the positions, 100 grs. of the water being measured out into its proper precipitating bottle for each of the six working days of the week, and corresponding to each of the eight positions.

During the second year samples were taken only from the surface and at the position near the east bank. The tabulated results of these measurements are given in Humphreys and Abbot's Report (edition of 1876, pp. 134, 417).

From the study of these results, Humphreys and Abbot drew the following conclusions:

"This table is fruitful in results. It establishes that the Mississippi water is not charged to its maximum capacity with sediment, because the distribution of the material is different from what must have place were this the case. Dupuit demonstrates that the power of suspension is due to the fact that the different layers of water are actuated by different velocities, and thus exert different pressures upon the different sides of the suspended atoms. Hence, the greater the difference in the velocity of consecutive layers, the greater will be the power of suspension. Now, it is conclusively proved in Chapter IV[1] that the change of velocity from layer to layer is, in horizontal planes, greatest near the banks and the least near the thread of the current; and in vertical planes, parallel to the current, the greatest near the bottom and surface, and the least at a point about 0.3 of the depth below the surface, where the absolute velocity has its maximum value. If, then, the water be either charged to its maximum capacity or overcharged with sediment, we must find the greatest amount near the banks and

[1] "Report on the Mississippi River." Humphreys and Abbot.

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near the surface and bottom, and the least amount near the thread of the current and near the layer 0.3 of the depth below the surface. If the water be undercharged, on the contrary, the distribution of sediment will follow no law, the amount at any point being fixed by the accidental circumstances of whirls, boils, etc., although, of course, there will be an accumulation of material near the bottom, where the suspending power is very much greater than elsewhere. Bearing these well-established principles in mind, an inspection of the preceding table must convince any one that the Mississippi water is undercharged with sediment, even in the low-water stage. A most important practical deduction may be drawn from this fact, namely, the error of the popular idea that a slight artificial retardation of the current, that caused by a crevasse, for instance, must produce a deposit in the channel of the river below it."

Sediment observations were also made at Columbus by the Mississippi survey from March to November, 1858, but, as only surface specimens were taken and no tabulated results give a means of comparison between the amounts in suspension near the banks and at the thread of the current, they can be of little service in the scientific study of the distribution of sediment in the cross-section of the river. Curves are shown, however, on Plates XII and XIII of their Report,[1] from which the relation between the mean velocity of the river and the corresponding mean amount of suspended matter at Carrollton and Columbus may be seen. The values from which these curves are plotted are given at page 417 of the same Report (edition of 1876).

From them Humphreys and Abbot are led to the same conclusion as before, *i. e.*, that the Mississippi water is not saturated with sediment, using the term in the sense in which it is used by M. Scipion Gras.[2]

Their line of reasoning is as follows: If the water be at all times charged with sediment to the maximum capacity allowed by its velocity, then the amount of sediment at different stages must vary proportionately with the mean velocity.

"At the date of highest water, both in 1851 and 1858, the river held in suspension but little more sediment per cubic foot than at dead low water. * * * Moreover, it will be seen that an analysis of the distribution of sedimentary matter held in suspension leads to the same

[1] "Report on the Mississippi River." Humphreys and Abbot.

[2] See page 249.

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conclusion, by establishing that the river is never charged to its maximum capacity of suspension."

Extreme care was taken in all these measurements in determining the amount of sediment in the sample obtained. It was shown that determinations of sediment must be made by weight and not by volume, as the latter method introduced discrepancies. These were due to the difference in density of the sediment, resulting from different methods of manipulation by various observers.

An extended series of measurements had been in progress on the Elbe, at Harburg, during the years 1837 to 1855, by Baurath Blohm.[1] The data obtained were minutely examined and formed the nucleus for a work treating of the subject in all its bearings. The early death of Herr Blohm prevented the publication of anything but the introductory part of the proposed book. Reference will be made to these observations later.

In 1871, M. Partiot published studies[2] on the movement of sands in the Loire, and enriched the knowledge of the subject by minute observations and extensive measurements. This monograph deserves especial mention, as it brings out strongly the importance of vortices and eddies in the suspending power of water.

Attention is called to the interaction of the suspended particles in changing their forms by friction, to the suspension and disintegration of the clays in the higher layers and their mixture with vegetable matter to form the rich alluvial deposits which settle on the summit of the shoals at the embouchure of the Loire. The slower moving sands are carried only intermittently in suspension.

Measurements are given to show that the quantities of sediment decrease toward the river mouth, and the interesting point is determined by experiment that the amount of silt varies not only with the height of the flood, with reference to others, but also with its own relative state. Increasing as the flood crest approaches, it reaches a maximum at its summit and descends to a lower point at the middle of the posterior slope than at the corresponding point of the anterior slope.

Partiot emphasizes the idea that the sands are only sustained by

[1] "Ueber die in fliessenden Wasser suspendirt erhaltenen Sinkstoffe." *Zeitschrift des Architekten und Ingenieur Vereins*, Hanover, 1867, pp. 240–297.

[2] "Mémoire sur les Sables de la Loire." M. Partiot. *Annales des Ponts et Chaussées*, I, 1871.

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eddies and vortices. He refers to experiments made at Nantes in 1869. Samples were taken at different depths at a point where the river was straight and free from eddies. Sand was not found in suspension, though when introduced 60 ft. above, in a surface velocity of 1.4 ft. per second, its presence was readily detected. At another point, where there was a marked eddy, grains of sand and mica were seen to surge to the surface and glitter in the sunlight, while grains of quartz were brought up in the receptacle from all depths. When the vortices were rapid, grains could even be taken in the hand.

The production of these vortices and eddies is attributed to the inequalities of the bottom, the solids deposited there, the deflecting action of concave banks and the action of floods. As the flood moves down a river in the form of an attenuated wave, the water flows down the front incline with an accelerated velocity. It overtakes the surface water down stream and flows over it, causing eddies. These grow greater as the crest of the wave is approached, since the fall increases, reach a maximum at the summit and decrease on the posterior portion, where the fall is decreased. This view is corroborated by the corresponding measurements of sediment in suspension.

The great velocities which these vortices reach in time of flood explain the movement of boulders, which could not be taken up by ordinary waters. M. Partiot calls to mind the lifting strength of whirlwinds as a parallel case. An interesting point is brought up in the reference to the action of ice in the movement of these solids. The sand grains and pebbles, as well as large stones, at times become frozen into the ice forming at the bed and banks of streams. With the least rise in the water this may become detached and carried to great distances.

It is to dragging rather than suspension that Partiot attributes the motion of sands in the Loire, and quotes some valuable researches made by M. Sainjon in this connection.

A body immersed in a moving liquid is subjected by the current to a thrust which may be expressed by

$$k \gamma F h = k \gamma F \frac{v^2}{2g}.$$

M. Sainjon takes the constant $k = 1.46$ for a prism, and $k = .60$ for a sphere, or as an average $k = 1$ for the particles making up gravels.

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The action of gravity upon this immersed body tending to roll it down stream is put equal to

$$(\gamma' - \gamma)\ V\left(\frac{i}{\sqrt{1+i^2}} - f\right)^1$$

Since i rarely reaches the value $\frac{1}{10}$ it is neglected and the approximate expression becomes

$$-V(\gamma' - \gamma)f,$$

whence the resultant force in the direction of the current becomes

$$P = \gamma F \frac{v^2}{2g} - V(\gamma' - \gamma)f$$

assuming $k = 1$.

To determine the value of f, M. Sainjon uses the results of the experiments of Dubuat. In this case the bottom velocities were determined at which the various materials ceased to be moved by the current[2], and at this point he considers that the approximate resultant force obtained above may be put $= 0$, *i. e.*,

$$\gamma F \frac{v^2}{2g} - V(\gamma' - \gamma)f = 0,$$

whence

$$f \frac{V}{F}(\gamma' - \gamma) = \gamma \frac{v^2}{2g},$$

---

[1] This value is inexact. The correct value is derived as follows:

Represent the resultant weight in water of the body rolling down the inclined river-bed by $W = V(\gamma' - \gamma)$. Let $\beta =$ the angle of the inclination of the bed.

The gravity component parallel to the bed $= W \sin. \beta$..... (1)

The normal component of $W = N = W \cos. \beta$.

The rolling friction $= F = N \tan. Q = W \cos. \beta \tan. Q$..... (2)

Therefore the resultant force acting is:

$$W \sin. \beta - W \cos. \beta \tan. Q.$$

or since $\tan. \beta = i$ and $\tan. Q = f$, this becomes

$$V(\gamma' - \gamma)\left(\frac{\tan. \beta}{\sec. \beta} - \cos. \beta \tan. Q\right) = V(\gamma' - \gamma)\left(\frac{\tan. \beta}{\sqrt{1 + \tan.^2 \beta}} - \cos. \beta \tan. Q\right)$$

$$= V(\gamma' - \gamma)\left(\frac{i}{\sqrt{1+i^2}} - f \cos. \beta\right).$$

By neglecting i, however, thereby virtually putting $\beta = 0$, this final expression reduces to the same form as M. Sainjon's approximate expression $-V(\gamma' - \gamma)f$ which is the one used. Therefore no results are vitiated.

[2] Sainjon is so quoted by Partiot ("Sables de la Loire," p. 32). In one case, at least, that of large sand, Dubuat states ("Traité d'Hydraulique," Paris, 1786, p. 94) that the velocity given is that at which the sand began to be moved as the bottom velocities were increased gradually from 3 up to 8 pouces per second.

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These results are tabulated as follows:

Kind of material.	Bottom velocity of current. Meters per second.	$\frac{v^2}{2g}$	$\gamma' - \gamma$	$\frac{V}{F}$	$\frac{V}{F}(\gamma' - \gamma)$	$\gamma \frac{v}{2g}$	f
Dark potter's clay.....	0.061	0.0003	1.64	.....	....	.....	....
Sand deposited by this clay.....	0.162	0.0013	.....	... . ..	....	.....	....
Coarse sharp yellow sand.	0.216	0.0024	1.36	0.002	0.0027	0.0024	0.88
Seine gravels: Size of anise seed.....	0.108	0.0006	1.545	0.001	0.0015	0.0006	0.40
Seine gravels: Size of peas...	0.189	0.0018	1.545	0.003	0.0046	0.0019	0.41
Seine gravels: Size of small beans	0.325	0.0054	1.545	0.0045	0.0069	0.0054	0.78
Rounded sea pebbles of an inch or more diameter.....	0.650	0.0215	1.614	0.018	0.0291	0.0215	0.74
Angular flints (size of a hen's egg).....	0.975	0.0484	1.250	0.045	0.0562	0.0484	0.86

The mean value of f is 0.68. Eliminating the two values, 0.40 and 0.41, so widely different from the others, the mean would be 0.80. Since 0.68 is approximately the tangent of the slope of a natural talus of ordinary earth, while wet sand and earth should have a greater cohesion, M. Sainjon chooses to use the value $f = 0.80$.

Taking the ratio of the bottom velocity in the Loire to the mean velocity at 0.7 (determined by measurements with a Woltmann's wheel), and assuming in general $\gamma' - \gamma = 1.50$, while $\frac{V}{F}$ equals two-thirds of the diameter for round forms and equals the diameter for angular ones, he computes the following table of velocity limits above which gravels will begin to be dragged.

Size of gravel. Diameter in meters.	Velocity at bottom. Meters per second.	Mean velocity. Meters per second.
0.0025	0.25	0.36
0.01	0.50	0.70
0.04	1.00	1.43
0.10	1.50	2.14
0.17	2.00	2.86
0.38	3.00	4.29
0.67	4.00	5.21

M. C. Lechalas published a memoir,[1] also in 1871, in which he takes exception to the theory attributing suspension to the phenomenon of relative velocities. He urges that this assumes flow in parallel fila-

[1] "Les rivières à fond de sable." *Annales des Ponts et Chaussées*, 1871. Published also, after revision, as an annex to Guillemain's "Navigation Intérieure—Rivières et Canaux." Tome I. Paris, 1885.

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ments which corresponds in no wise to movements under great velocities. His explanation attributes suspension to repeated shocks from the molecules of water moving more rapidly than the suspended body and to the action of eddies caused by the banks and bottom. He also calls attention to the fact that the variations in velocity in large rivers are much more rapid in the vertical than in the horizontal direction. To these rapid vertical variations he attributes the formation of some horizontal vortices.

The body of this valuable paper is devoted to an attempt to derive numerical results for the values of the mean depth, mean velocity and fall, in alluvial rivers, which will follow the contraction of its width, throughout a given length, by training walls. Certain parts, however, of M. Lechalas' work have a direct bearing on the relations between velocity and movement of sedimentary matter. It will be seen that he lays stress on the distinction between transportation by dragging and by suspension.

Referring to Dubuat's experiments, he expresses the excess of pressure on the up-stream face of an immersed body as proportional to the square of the velocity of the water surrounding the body, *i. e.*, for sand grains on a river bed,

$$\text{Thrust of the moving water} = a^1 v_o^2,$$

where a is a constant which varies with the dimensions, form and position of the grains of sand.

The resisting force of the sands of the Loire is put equal to $a\ 0.25^2$ since they are not transported until the bottom velocity reaches 0.25 meter.[2]

The resultant force—

$$P = a\ (v_o^2 - 0.25^2)$$

is put equal to the mass of the particle multiplied by its acceleration[3] parallel to the direction of v_o. Measuring the velocity v_o in the direction of the axis of the river, M. Lechalas considers this resultant

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[1] $a\ v_o^2$ corresponds to the expression $\frac{k\ \gamma\ F}{2\ g} v^2$ used by M. Partiot, quoted on page 253.

[2] Compare table quoted from Sainjon at page 369. This refers to sands not already compacted by the continued action of currents of velocity too slight to transport the grains, but yet sufficient to increase the resisting power of the surface lamina.

[3] M. Lechalas has used the word *vitesse* here. It must, however, be meant for acceleration.—See "Navigation Intérieure," Guillemain, Annexes—Rivières à fond de sable p. 489.

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force proportional to the discharge of sand in the river and puts—

$$a\,(v_o^2 - 0.25^2) = b\,d$$

or

$$d = \frac{a}{b}\,(v_o^2 - 0.25^2) = m\,(v_o^2 - 0.25^2) = m\,(v_o^2 - 0.06)$$

where d represents the discharge of sand per unit of width.

The value of m is to be determined by observation, and in this way a correction made for the use of v_o, the absolute velocity of the water at the bed, instead of the relative velocity of the water and the solid particle. When the velocity v_o becomes greater than a certain value, the particles are lifted and cease to roll on the bottom. The term $-\ m\ 0.06$ then disappears and

$$d = m\ v_o^2$$

for particles suspended immediately above the bottom.

The advancement of the crests of the sand bars measured by M. Sainjon in the Loire gives a method of determining the value of d for corresponding values of surface velocity v_1, and also a means of comparing this advancement with the corresponding velocities of the current v_o at the bottom.

TABLE OF OBSERVATIONS ON ADVANCEMENT OF CRESTS OF SAND BARS.

Observed surface velocity in meters per second.	Height in meters of the crest above the down-stream bed.	Displacement of the crest in hundred-thousandths of a meter per second.	
		Observed for a lapse of many days.	Computed.
0 58	0.900	3.0	3.0
0.64	0.300	3.3	3.9
0.73	.....	5.1	5.5
0.75	0.782	6.3	5.9
0.81	0.967	6.7	7.1
0.81	.....	7.5	7.1
0.83	0.760	7.6	7.5
1.00	0.963	10.5	11.6
1.016	0.920	12.4	12.0
1.016	0.580	12.0	12.0
1 03*	0.487	6.2	12.35
1.05	0.612	7.0	12.9
1 11	1.198	5.8	14.6
1.13	0.650	8.7	15.2
1.33	0.950	5.6	21.6

* In the table quoted above by Lechalas from Sainjon the following note appears: "It is wrong to suppose the co-existence of rolling and suspension for the velocities 1.03 m. * * * * The absolute lack of accord with the law of advancement up to $v_1 = 1\ 016$ can only correspond to a complete transformation of the method of transport. If there was a mixed period extending between the surface velocities 1.03 and 1.33, the calculated velocities would only differ gradually from the observed ones. Instead of that, we see that beyond $v_1 = 1.03$ the observed movements are not more than half the computed ones. This remaining advancement is explained by the deposit of sands held in suspension—a deposit caused by the sudden diminution of velocity below the crest. No trace of the mixed period remaining when the surface velocity reaches 1.03 m., it is probable that it was about over when the value reached 1.02 m."

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The computed values of the displacement of the crest given in the last table are derived from the formula—

$$\text{Displacement} = 0.00013\ (v_1^2 - 0.11),$$

in which M. Sainjon expresses the rate of advancement of the crest so long as the surface velocity does not exceed 1.016 m. per second.

In discussing this formula M. Lechalas calls attention to the fact that this displacement becomes o for $v_1 = \sqrt{0.11}$, which virtually[1] corresponds to a bottom velocity of 0.25 m., and that one could write

$$d = m\ (v_1^2 - 0.11),$$

by giving to m the value 0.00013 × the mean height of the crests in the last table corresponding to the surface velocities from 0.58 m. up to 1.016 m. This mean height is equal to 0.77 m. and the product gives $m = 0.0001$.

Whence

$$d = 0.0001\ (v_1^2 - 0.11) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (0)$$

He objects, however, to this form because v_1, the surface velocity, will vary with the depth of the stream and so introduce another variable.

Since the bottom velocity v_o ought not to vary widely, he prefers his own equation

$$d = m\ (v_o^2 - 0.06) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

to equation (0), preceding, as given by M. Sainjon.

Referring to the preceding table, he adopts the value 1.016 m. as the upper limit for surface velocities at which dragging occurs. Up to this velocity the equation (1) will be used, and beyond this limit the equation

$$d = m\ v_o^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

will be used to express the relation between solid discharge and velocity at the bottom.

It remains to find the value of v_o which corresponds to $v_1 = 1.016$ m. M. Lechalas does this by using the formulas of Darcy and Bazin

$$\frac{Z\,i}{v^2} = 0.00028 + \frac{0.00035}{Z} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

$$\text{and } v_1 = v + 14\sqrt{Z\,i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (b)$$

By combining these two equations,

$$\frac{v_1}{v} = 1 + 14\sqrt{\frac{Z\,i}{v^2}} = 1 + 14\sqrt{0.00028 + \frac{0.00035}{Z}} \quad \ldots\ldots (c)$$

[1] M. Sainjon (see page 369) considers $v_o = .7\ v_1$; $v_1 = \sqrt{0.11} = 0.331$; $v_o = 0.7 \times 0.331 = 0.23$ m. per second.

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Substituting the value $v_1 = 1.016$ m. and the values

$$Z = \text{mean depth} = 0.5 \text{ m.}$$
$$= 1.0 \text{ m.}$$
$$= 2.0 \text{ m.}$$

he derives the corresponding values

$$v = 0.71, 0.75 \text{ and } 0.78.$$

By combining Darcy and Bazin's equations

$$v_1 = v + 14\sqrt{Z\,i} \text{ and}$$
$$v_1 = v_o + 24\sqrt{Z\,i}$$

he obtains

$$v_o = v - 10\sqrt{Z\,i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (d)$$

which combined with (*a*) gives

$$\frac{v_o}{v} = 1 - 10\sqrt{0.00028 + \frac{0.00035}{Z}} \quad \ldots\ldots\ldots\ldots\ldots (e)$$

Substituting the values of v which correspond to the assumed values of Z, the values of v_o are obtained—

$$v_o = 0.49, 0.56 \text{ and } 0.61.$$

M. Lechalas adopts a mean of these values, $v_o = 0.55$ m., as the upper limit of the bottom velocity corresponding to transport by rolling on the bed of the Loire. Since the range of velocities for which the table on page 257 gives indications of a combined mode of transport occupies such a small part of the velocity scale (from $v_1 = 1.016$ m. to $v_1 < 1.03$ m.), he assumes the same value, $v_o = 0.55$, as the lower limit of the bottom velocity corresponding to transport by suspension.

To show more clearly the actual relation of these velocity limits to the variables of the current, an ideal canal is assumed, of constant width and of a flow equal to 3 cu. m. per unit of width, so that

$$b = 1 \text{ and } q = 3.$$

Equations (*a*) and (*d*) preceding combined with the equation

$$q = Z\,v,$$

which expresses the definition of liquid discharge when b is put equal to 1, give, for $v_o = 0.55$ m.,

$$i = 0.000035,\ Z = 4.50 \text{ m., and } v = 0.67 \text{ m.}$$

and, for $v_o = 0.25$ m.,

$$i = 0.000003,\ Z = 10.00 \text{ m. and } v = 0.30 \text{ m.}$$

To express these results in the words of M. Lechalas:

"A bed of regular width, filled with sand which is not renewed, and which lies at an inclination exceeding a certain limit, receives a

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discharge of 3 m. of water per second per unit of width. After a length of time greater or less, according to the fall and the length of the canal, a state of unstable equilibrium establishes itself. The mean depth is then 4.50 m., the mean velocity 0.67 m., the fall 3.5 cm. per kilometer, and the bottom velocity 0.55 m.

"The sand, however, is still transported, but in quantities smaller and smaller each second. After a considerable time a new state of equilibrium is established. This is final; it corresponds to a mean depth of 10 m., a mean velocity of 0.30 m., a fall of 3 mm. per kilometer, and a bottom velocity of 0.25 m. Although these computations apply only to an ideal channel, yet they are of interest as showing what an important role is played by the consideration of these velocity limits in the study of alluvial rivers."

Returning to the equations—

$$d = m\,(v_o^2 - 0.06) \text{ for } 0.25 \text{ m.} < v_o < 0.55 \text{ m.} \quad \ldots\ldots\ldots\ldots (1)$$

and

$$d = m\ v_o^2 \qquad \text{for } v_o > 0.55 \text{ m.} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

M. Lechalas uses the following method to determine the value of m. By combining equations (c) and (e) of pages 258 and 259.

$$\frac{v_1}{v_o} = \frac{1 + 14\sqrt{0.00028 + \frac{0.00035}{Z}}}{1 - 10\sqrt{0.00028 + \frac{0.00035}{Z}}} \quad \ldots\ldots\ldots\ldots\ldots (f)$$

which gives the ratio between the surface and bottom velocities in the artificial canal used by Darcy and Bazin.

The bottom velocity ought to be less dependent upon the depth than that at the surface. If a formula is expressed in terms of the bottom velocity, it may properly be transformed into terms of the surface velocity and mean depth, or of mean velocity and mean depth. On the other hand, when the formula is in terms of the surface velocity, and it is desired to express it in terms of the bottom velocity, it is necessary to assume the ratio $\frac{v_1}{v_o}$ a constant for all values of the mean depth. This introduces an approximation unavoidable without a new series of observations.

Assuming $Z = 1$ meter[1] in equation (f)

$$\frac{v_1}{v_o} = 1.80.*$$

[1] The mean of the values of Z used to obtain the critical value $v_o = 0.55$ m., and hence the most consistent value to use in determining m.

* For $Z = 3$ m., $\frac{v_1}{v} = 1.60$.

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Equation (0) page 258, which is based upon M. Sainjon's empirical formula, may be considered reasonably accurate for the range of velocities for which it is intended, as can be seen from a study of the computed results in the table of page 257.

Equation (0) and equation (1) of page 258 may now be written

$$d = 0.0001\ (v_1^2 - 0.11) = m\ (v_o^2 - 0.06)$$

and, by introducing the approximate value $v_1 = 1.80\ v_o$ from page 260, the value found

$$m = \frac{0.0001\left(\overline{1.8\,v_o}^2 - 0.11\right)}{v_o^2 - 0.06}$$

For $v_o = 0.50$ m. per second.

$$m = 0.00037^1$$

and M. Lechalas' equation (1) becomes

$$d = 0.00037\ (v_o^2 - 0.06) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and (2) becomes

$$d = 0.00037\ v_o^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The objections to the introduction of the uncertain value of the ratio $\frac{v_1}{v_o}$ in obtaining these final equations are all admitted, but M. Lechalas maintains that if a numerical coefficient can be used when the discharge d is expressed in terms of v_1 at the surface,[2] one can be much more reasonably used when the equation is in terms of the bottom velocity v_o.

The years 1874 to 1879 marked the arousal of a great popular interest in the United States in the question of silt movements in the Mississippi. The bitter controversy between the Government engineers and Captain James B. Eads with his associates over the improvement of the mouth of the river need not be entered into here. Suffice it to say that the many spirited articles written on the subject during those years were not of great scientific value and left the knowledge of the distribution of the sediment in the river in the same state of incompleteness in which it was left by the report of Humphreys and Abbot in 1861.

Mr. Eads states his views in a letter[3] of March 15, 1874, with reference to these sediment movements in the following words:

[1] For values of v_o between 0.40 m. and 0.55 m., the corresponding values of m range between 0.00041 and 0.00036.

[2] As is done in the equation (0) based on M. Sainjon's formula; displacement $= 0.00013 \times (v_1^2 - 0.11)$.

[3] To William Windom, United States Senate, Chairman of Committee on Transportation Routes to the Seaboard.

See "The Mississippi Jetties," p. 28, E. L. Corthell, New York, 1881.

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"By far the greatest portion is, however, transported in suspension. The amount of this matter and the size and weight of the particles which the stream is enabled to hold up and carry forward depend wholly upon the rapidity of the stream, modified, however, by its depth. * * * A certain velocity gives to the stream the ability of holding in suspense a proportionate quantity of solid matter and when it is thus charged it can sustain no more. * * * The fact that a given current will keep in suspension a corresponding quantity of solid matter; that at a less velocity a portion of it will be deposited and taken up again at a greater, is fully recognized in experimental science and has been extensively made use of for analysis of soils. An eminent investigator of this subject, Prof. E. W. Hilgard, of the University of Michigan, now of the University of California, Oakland, Cal., has classified silts according to the different velocities at which they deposit.[1] This independent line of research fully confirms the view herein advanced in explanation of the phenomena presented through the alluvial bed of the Mississippi."

Gen. A. A. Humphreys, Chief of Engineers, expresses his views in a report[2] to the Secretary of War, dated April 15, 1874, in the following words:

"It has been recently stated by a civil engineer,[3] in a pamphlet concerning the improvement of the mouths of the Mississippi River by jetties, that the amount of sedimentary matter carried in suspension by the Mississippi River is in exact proportion to the velocity of its current; and that, as a given velocity of current will keep in suspension a corresponding quantity of solid matter at a less velocity a certain portion of it will be dropped. * * * The first statement is in direct conflict with the results of the long-continued measurements made upon the quantity of earthy matter held in suspension by the Mississippi River at Carrollton, near New Orleans, and at Columbus, 20 miles below the mouth of the Ohio, one of the chief objects of which was to determine this very question, whether any relation existed between the velocity and the quantity of earthy matter held in suspension. These results prove that the greatest velocity does not correspond to the greatest quantity of earthy matter

[1] *American Journal of Science* III, VI, 337.

"The classified table of Prof. Hilgard gives the relative velocities created in a mechanical contrivance made for test purposes in a laboratory in which coarse sand is dropped at a certain velocity of the machine, which may be represented in nature as a current of about 2.5 ins. per second; the finest sand when the current is 0.3 of an inch per second; the coarsest silt when the velocity is 0.14 of an inch per second; the finest silt when the velocity is 0.02 of an inch per second."

"Report of Chief of Engineers, U. S. Army," 1874, Part I, p. 865.

[2] "Report of Chief of Engineers, U. S. Army," 1874. Part I, p. 863.

[3] James B. Eads.

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neld in suspension; on the contrary, at the time of the greatest velocity of the current at Carrollton, the river held in suspension but little more sediment per cubic foot than when the velocity was least. When the quantity of earthy matter held in suspension was greatest the velocity was 2 ft. per second less than the greatest velocity, the quantity of earthy matter in the one case being three times as great as in the other. We find at another time, when the velocity was one-half the greatest velocity the quantity of earthy matter held in suspension was double the amount. Again, we find the quantity of earthy matter in suspension the same, the velocity in the one case being 6.75 ft. per second, and in the other, 1.5 ft. per second.

I.—Carrollton, 1851.

Date.	Weight in grains of sediment in 1 cu. ft. of water.	Mean velocity of river in feet, per second.	Remarks.
February 20th	450	6.5	
March 20th	200	6.2	
April 15th	150	5.6	
May (last week of)	100	3.75	
June 20th	650	4.3	
July 10th to 30th	450	4.8	
August 1st to 20th	450	From 4.8 to 3.5	Change in velocity regularly decreasing, while suspended matter remains the same.
September 8th	300	3.0	
October and November	100	1.75	
December	175	1.85	
January 20th, 1852	400	2.75	

II.—Columbus. Twenty Miles below the Mouth of the Ohio, 1858.

Date.	Weight in grains of sediment in 1 cu. ft. of water.	Mean velocity of river in feet per second.	Remarks.
April 1st	300	7.00	
April 10th	300	5.25	
April 25th	450	7.25	
May 1st	300	7.50	
May 10th	300	5.75	
May 22d	160	6.75	
June 16th	330	8.25	
July 16th–17th	650	3.75	
August 2d	350	4.75	
August 9th	250	4.00	
September 2d	600	2.50	
September 9th to 23d	200	2.25	
October (all of)	200 to 100	1.50	Uniform decrease in amount of sediment, the velocity remaining the same.

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"The tables (on page 263) illustrating what has just been said, have been prepared from the report on the Mississippi River. The figures given express the conditions not only on the day noted, but on several successive days.

"It is to be remarked that the investigations respecting the sediment in suspension show that the quantity depended on the river from which the volume of discharge was at the time chiefly derived.

"The cross-sections, both at Carrollton and Columbus, remained unchanged during the above observations."

In order to define still more clearly the position of General Humphreys on this question, the following quotation is made from his report of 1875:[1]

"It has been sometimes stated that every velocity of current is capable of carrying in suspension a certain fixed quantity of earthy matter, and that the water of a muddy river is always thus charged with the maximum quantity of earthy matter it can carry. * * * But this assumption as to the carrying power of currents is utterly disproved by long series of exact measurements upon the Mississippi River. * * * These measurements upon the quantity of earthy matter suspended in the Mississippi River show that at no time has the water been so heavily charged with it that the current could not carry it along in suspension to the same extent as it did when the quantity of earthy matter was least; and they further show that the current of the Mississippi River, when most feeble, can carry in suspension the greatest quantity of suspended earthy matter found in it to the same extent that it can carry the least quantity found in it.

"It was undoubtedly the observation of facts similar to these that led to the conclusion, entertained by some, that the suspending power of the current of a river did not depend upon its absolute rate of motion, but upon the difference of velocity between the adjoining fillets of water. There is good reason to conclude that this is one of the causes or sources of the suspending power of a stream.

"This proposition, therefore, respecting certain velocities of current always carrying certain fixed quantities of earthy matter, and always adjusting those quantities according to its own variations of strength is so entirely disproved by facts that it will not be considered again."[2]

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[1] Annual Report of Chief of Engineers, U. S. A.," 1875, Part I., pp. 959–975. Reprinted in Humphreys and Abbot's "Report on the Mississippi River." Edition of 1876. Appendix M, p. 684.

[2] Those readers who wish to go farther into the details of this somewhat amusing controversy are referred to Humphreys and Abbot's Report on the Mississippi River, Edition of 1876. Appendices.

Review of same by James B. Eads, M. Am. Soc. C. E., in *Van Nostrand's Engineering Magazine*, Vol. XIX, 1878, pp. 211–229.

Answer to Mr. Eads' attack by General Henry L. Abbot, *Van Nostrand's Engineering Magazine*, January, 1879, Vol. XX, pp. 1–6.

Answer to General Abbot by Mr. Eads, *Van Nostrand's Engineering Magazine*, Vol. XXI, 1879, p. 154.

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An article by Mr. G. K. Gilbert,[1] upon the erosion of the Colorado cañons, appeared in the *American Journal of Science*, July and August, 1876. While subject to some criticism, it may be regarded as a most valuable contribution to the knowledge of the laws of transport of solid bodies by water currents. It is believed that Mr. Gilbert is the only writer who has called attention to the fact that the same expenditure of energy will transport a greater weight of fine particles than of coarse ones of the same density.

A series of observations was conducted by Assistant Engineer J. B. Johnson at Helena, on the Mississippi River, in 1879.[2] Longitudinal and transverse soundings were made to determine the existence and movement of sand waves in the river bed, and the results plotted[3] so as to show clearly the presence of these undulations. From the observations Mr. Johnson deduces the following facts:

"Average length of waves from crest to crest, about 100 m.

"Extreme length of waves from crest to crest, about 150 m.

"Average height of waves from crest to valley, about 5 ft.

"Extreme height of waves from crest to valley, about 8 ft.

"Average velocity of motion of crest, 5.41 m. per day.

"These results were obtained in a depth of water varying from 13 to 30 ft. The stage of the river varied from 12 to 18 ft. above low water at Helena. The waves decreased in size for a falling river and *vice versa*. Their rate of motion down stream is a function of the velocity of the water. They do not extend from bank to bank at Helena but disappear about 200 m. from each shore, covering about 1 000 m. of the cross-section of the river."

Sediment measurements were made by the same party from March 1st to June 18th, 1879, and deserve special mention because of the introduction of an improved sediment can[4] for bringing up specimens from the bottom.

Samples were taken each day from the surface and 1 ft. above the bottom at points one-fourth and three-fourths of the distance across the river. Proportions of sediment were determined by weight in the later experiments and the mean velocity of the river was determined by floats upon five occasions during the extent of the observations.[5]

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[1] See digest in *Engineering News*, August 19th, 1876.

[2] See "Report of Chief of Engineers, United States Army," 1879, Part III, pp. 1963–1970.

[3] See Plate I, p. 1966. "Report of Chief of Engineers," 1879, Part III.

[4] For sketch, see "Report of Chief of Engineers, United States Army," 1879, Part III, p. 1965.

[5] The tabular result of these observations is given at p. 1969 of above Report.

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Simultaneous observations of a like nature were conducted at St. Louis[1] by R. E. McMath. They are more satisfactory in that they offer a slight opportunity for study of transverse distribution of sediment. Both sets give velocity measurements.

The most extended set of observations published upon sediment movements and sand waves are those instituted by the Mississippi[2] and Missouri[3] Commissions in 1879–1881. These were made at St. Louis, Carrollton, Prescott, Winona, Clayton, Hannibal, Grafton and St. Charles. They are wide enough to put at rest certain debated questions, but yet fail in several points to be completely satisfactory—notably in failing to give data on horizontal distribution. These measurements will be again referred to.

Major Allan Cunningham made a series of observations on the Ganges Canal[4] to determine the amount of sediment carried and its distribution in the cross-section.

A tube 12 ft. long, open at both ends, was thrust down vertically from a floating boat until the bottom was reached. It was then closed at the bottom, by a lid worked by a spring, and the column of water, extending from bed to surface, carefully separated from its sediment by decantation and filtration.

This sediment, when weighed, gave a result, called silt-density, which represented the average density in the vertical examined.

To determine the distribution of silt, two collections were made, by the method indicated, at each of nine points in the width of the canal at two different cross-sections. Each set was completed as rapidly as possible. The mean silt velocity past each vertical was computed by multiplying this silt density by the corresponding mean velocity. Captain Cunningham then plotted three transverse curves on a common base, using as ordinates the silt density, the mean silt velocity, and the mean velocity past each of the nine verticals. From the want of relative connection between these curves he concludes that in the Ganges Canal there is no close relation between the silt and the velocity at different parts of the channel, and that the silt density at any point varies from instant to instant.

Van Nostrand's Engineering Magazine, 1883, p. 33.

2 "Report of Chief of Engineers United States Army," 1883, III.

3 "Report of Chief of Engineers, United States Army," 1887, IV.

4 "Roorkee Hydraulic Experiments," Roorkee, 1881, Chap. XXIV. Abstracted and discussed in *Proceedings* of the Institute of Civil Engineers, 1882, Vol. LXXI, pp. 1–94. Same reproduced in *Van Nostrand's Engineering Magazine*, April and May, 1883.

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In continuation of these measurements, 73 collections were made at four of the cross-sections, the depth and velocity at two of them being very different. These results led to the conclusion that the mean silt density in no way depended upon the depth or velocity in this canal, but rather upon the state of the supply water from the Ganges.

The best known formula for the determination of the size of particles dragged by a current of a given velocity is that proposed by Mr. Wilfred Airy, and derived by him as follows:[1]

Let a = the length of the largest cube the current could move.

Then weight of cube $= \gamma' a^3$ (γ' const.).

Friction of cube on bed of river $= f' \gamma' a^3$ (p const.).

Total pressure of current on exposed face of cube $= k a^2 v_o^2$ (k const.)

For equilibrium—

$$f' \gamma' a^3 = k v_o^2 a^2$$

whence

$$a = \frac{k}{f' \gamma'} v_o^2$$

therefore the weight of the largest cube which a current with a bottom velocity v_o could move would be

$$\gamma' a^3 = \gamma' \left(\frac{k}{f' \gamma'} v_o^2 \right)^3 = \frac{k}{f'^3 \gamma'^2} v_o^6$$

If G' and G'' were the weights of cubes of silt, etc., which could just be moved by currents of bottom velocities v_o' and v_o'' respectively, then

$$\frac{G'}{G''} = \left(\frac{v_o'}{v_o''} \right)^6 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

or, numerically:

If v_o is increased by $\frac{1}{3}$ of itself, it will move particles of twice the weight, since $\left(\frac{1}{1\frac{1}{3}} \right)^6 = \frac{1}{2}$; and if the velocity v_o is doubled, it will move particles of 64 times the weight, since $\left(\frac{1}{2} \right)^6 = \frac{1}{64}$.

[1] See condensed description in *Proceedings* of the Institution of Civil Engineers, Vol. 32, p. 25. Notation changed.

See Church's "Mechanics of Engineering," p. 331.

A formula, showing that the scouring power of a natural stream is proportional to the seventh power of the velocity, is said to have been proposed about 1855, by W. Hopkins, of Cambridge, England.

See Baldwin Latham in *Proceedings* of the Institution of Civil Engineers, Vol. 32, p. 43.

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Mr. Henry Law shows this formula to be also applicable to the case of a cube rolled along instead of sliding, and to be true for a sphere as well as a cube.[1] His proof follows:

The moment of resistance of the cube to turning about its edge is

$$\gamma' a^3 . = \frac{a}{2} = \gamma' \frac{a^4}{2}$$

The turning moment of the thrust of the current is

$$k a^2 v_o^2 . \frac{a}{2} = \frac{k a^3 v_o^2}{2}$$

At the instant of turning the equation of equilibrium gives

$$\gamma' \frac{a^4}{2} = \frac{k a^3 v_o^2}{2}$$

whence

$$a = \frac{k}{\gamma'} v_o^2$$

Following the same process used by Airy above, this leads to his formula (1).

In the case of spheres, assume each one to be resting upon three others.

Weight of sphere $= \frac{4}{3} \pi r^3 \gamma'$.

Let r sin. β = lever arm of weight about point of turning.

Then moment of resistance to turning is

$$\frac{4}{3} \pi r^3 \gamma' . r \text{ sin. } \beta = \frac{4}{3} \pi \gamma' r^4 \text{ sin. } \beta$$

Thrust of the current $= k \pi r^2 v_o^2$.

Its lever arm about the point of turning would be r cos. β.

Then the turning moment due to the thrust would be

$$k \pi r^3 v_o^2 \text{ cos. } \beta.$$

For impending motion, the equation of equilibrium gives

$$k \pi r^3 v_o^2 \text{ cos. } \beta = \frac{4}{3} \pi \gamma' r^4 \text{ sin. } \beta$$

whence

$$r = \frac{3}{4} \frac{k}{\gamma'} v_o^2 \text{ cot. } \beta.*$$

This again leads to Mr. Airy's equation (1) as above, β being constant as r varies.

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[1] See *Proceedings* of the Institution of Civil Engineers, Vol. 82, pp. 29-30. Notation changed.

* Through an oversight, Mr. Shaw has obtained an incorrect numerical coefficient for this last equation in having used, for the value of the section of the sphere normal to the current, $\frac{1}{2} \pi d^2$ instead of $\frac{1}{4} \pi d^2$. It is corrected here.

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Mr. Shaw then concludes that the weight of particles moved by a current, whether cubes or spheres, and whether the action be sliding or rolling, will vary as the sixth power of the mean velocity of the current impinging on them, if cohesion between the particles be disregarded.

M. J. Thoulet, Professor of Science, at Nancy, published in 1884 the results[1] of some experiments made to determine the force required to keep particles of different sizes and densities suspended in water.

The apparatus used consisted of a glass tube placed in a vertical position and connected at its lower end by a rubber tube with a stopcock to regulate the velocity of a water-current ascending through the glass tube. The water was led away by a waste-pipe connected near the top, and the velocity for each experiment determined from the weight of water flowing. The details of the experiments were carried out with scientific exactitude.[2]

M. Thoulet computed the mean velocity of the current required in tubes of four different diameters (2.2, 4.775, 6.75 and 8.0 mm.) to hold unmoved, at a fixed point, spheres of different sizes and densities. These spheres were lead bullets of different calibers and balls of wax containing, in their interior, grains of tin, lead or copper. Their sphericity was tested under the microscope, and in all cases they were kept at the specified height for a length of time not less than 30 seconds.

From his results M. Thoulet has computed a table giving in millimeters per second the velocities of vertical currents of water capable of holding in suspension, at a fixed height in the tube, spherical grains of known radii and of given densities. These radii vary from 1 to 2.5 mm. and the densities from 1.5 to 4. The table[3] also gives, in milligrams, the thrust of the current against the grain.

This thrust is equal to the resultant weight of the grain immersed in water, *i. e.*,

$$\tfrac{4}{3}\,\pi\, r^3\,(\gamma' - 1) = \text{thrust}$$

and M. Thoulet has computed the values corresponding to the different values of r and γ' from this formula.

Making the assumption of spherical grains in a stream bed, he considers that each one may be regarded as resting on three others, and

[1] *Annales des Mines*, 1884, I, pp. 507–530. For digest, see *Annales des Ponts et Chaussées*, 1885, I, pp 492–500.

[2] See description in *Annales des Mines*, 1884, I, pp. 507–530.

[3] The same, p. 521. See p. 312 of this paper.

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shows graphically that for a horizontal movement a force will be required sufficient to move the grain up a slope of about 37°.[1]

This force $= \frac{4}{3} \pi r^3 (\gamma' - 1)$ sin. 37°.

By referring to his table M. Thoulet determines the bottom velocities required to exert a force equal to that demanded by this formula for the three cases given below.

Material.	Diameter of grains in millimeters.	Velocity required in millimeters per second.
Coarse mud	0.40	40 00
Fine sand	0.70	59.68
River sand	1.70	109.58

M. Vauthier, in a valuable paper[2] before the French Association for the Advancement of Science, in 1884, developed mathematical expressions for the velocity, at any instant, of a solid body falling through a liquid, and for the path described in a given time.

His method consists in writing the accelerating force equal to the mass multiplied by the acceleration of the body, assumed to be a sphere.

Accelerating force = weight of body — resisting force

$$= \frac{4}{3} \pi r^3 (\gamma' - 1) - \pi r^2 k \frac{v^2}{2g}$$

When the motion has become uniform the accelerating force will be zero, and one may write for this case.

$$\frac{4}{3} \pi r^3 (\gamma' - 1) - \pi r^2 k \frac{v'^2}{2g} = 0 \quad \ldots\ldots\ldots\ldots \quad (1)$$

whence,

$$v' = 2 \sqrt{\frac{2}{3} \frac{r g}{k} (\gamma' - 1)} \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

where v' represents the limiting velocity, after which motion is uniform.

For any stage of the motion

$$\frac{4}{3} \pi r^3 (\gamma' - 1) - \pi r^2 k \frac{v^2}{2g} = \frac{4}{3} \frac{\pi r^3}{g} \gamma' \frac{dv}{dt} \quad \ldots\ldots \quad (3)$$

$$= \text{mass} \times \text{acceleration.}$$

[1] This is the value of the angle β in Mr. Shaw's analysis preceding.

[2] "De l'entrainement et du transport, par les eaux courantes, des vases, sables et graviers."

L. L. Vauthier, "Mémoires de l' association française pour l' avancement des sciences," Blois, September 8, 1884.

Abstracted at length in the "Mémoires de la Société des Ingénieurs Civils de France," 1885, 2. pp. 29–35.

General results also given in *Engineering News*, November 1, 1884, p. 211.

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Subtracting (1) from (3) and simplifying

$$k\,(v'^2 - v^2) = \frac{8}{3}\,r\,\gamma'\,\frac{d\,v}{d\,t}.$$

Separating the two variables

$$\frac{3}{8}\,\frac{k}{r\,\gamma'}\,d\,t = \frac{d\,v}{v'^2 - v^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

Since $v = o$ for $t = o$ one may write:

$$\int_o^t \frac{3}{8}\,\frac{k}{r\,\gamma'}\,d\,t = \int_o^v \frac{d\,v}{v'^2 - v^2}$$

Integrating by partial fractions

$$\frac{3}{8}\,\frac{k}{r\,\gamma'}\,t = \frac{1}{2\,v'}\left(\int_o^v \frac{d\,v}{v' + v} - \int_o^v \frac{-\,d\,v}{v' - v}\right)$$

$$= \frac{1}{2\,v'}\log_e\left(\frac{v' + v}{v' - v}\right)$$

whence,

$$e^{\frac{3}{4}\,\frac{k}{r\,\gamma'}\,v'\,t} = \frac{v' + v}{v' - v}.$$

Putting

$$N = \frac{3}{4}\,\frac{k}{r\,\gamma'}\,v' \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

and transforming

$$v = v'\,\frac{e^{N\,t} - 1}{e^{N\,t} + 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

and, since $d\,s = v\,d\,t$

$$d\,s = v'\,\frac{e^{N\,t} - 1}{e^{N\,t} + 1}\,d\,t \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

whence, by integration, since when $s = o$, $t = o$:

$$s = v'\left[t - \frac{2}{N}\log_e 2 + \frac{2}{N}\log_e\left(1 + \frac{1}{e^{N\,t}}\right)\right]^1 \quad \ldots\ldots \quad (8)$$

---

[1] Put $e^{Nt} - 1 = u$ and $e^{Nt} + 1 = u + 2$.

Then $e^{Nt} = u + 1$. Differentiating, $du = e^{Nt}\,N dt$,

or $$dt = \frac{du}{N\,e^{Nt}} = \frac{1}{N}\,\frac{du}{u + 1}.$$

Substituting these values in (7) above

$$ds = v'\,\frac{u}{u + 2}\,\frac{1}{N}\,\frac{du}{u + 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (a)$$

Separating into partial fractions by the method of indeterminate coefficients (cf. Osborne's "Differential and Integral Calculus," p. 189).

$$\frac{u}{(u + 1)\,(u + 2)} = \frac{2}{u + 2} - \frac{1}{u + 1}$$

Substituting this value in (a) and integrating

$$\frac{N}{v'}\,s = 2\int\frac{du}{u + 2} - \int\frac{du}{u + 1} + C, \text{ or}$$

$$\frac{N}{v'}\,s = 2\log_e(u + 2) - \log_e(u + 1) + C \quad \ldots\ldots\ldots\ldots \quad (b)$$

(*Footnote continued on next page.*)

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Assuming $k = 0.5$ and $\gamma' = 2.0$ for a mean of the particles moved in river beds, and $g = 9.8088 = 10$, approximately, M. Vauthier obtains from (2) and (5) (for meter measure).

$$v' = 5.16393 \sqrt{2r} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

$$N = \frac{1.93648}{\sqrt{2r}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

This last equation shows that for particles of slight diameter, the value of N, and consequently of e^N, is very large.

Writing equation (6) in the form

$$v = v' \frac{1 - \frac{1}{e^{Nt}}}{1 + \frac{1}{e^{Nt}}}$$

it is at once seen that the fraction in the second member rapidly approaches the value 1 as the diameter of the particle is decreased, *i. e.*, as N approaches ∞. Therefore v approaches v' asymptotically, and at the limit the two will be equal.

For the same reason given above the transcendental term in equation (8) will be so small as to be negligible for particles of slight diameter.

By numerical substitution in equations (9), (10), and (8). M. Vauthier shows that, for a block as large as 1 m. in diameter, at the end of the first second the velocity will be only about ⅔ of the velocity limit v', and the transcendental term in (8) will be too large to be neglected, but that, with succeeding seconds, it tends rapidly to approach the value v', while the transcendental term tends rapidly toward 0.

When $s = 0$, $t = 0$, and hence $u = 0$, since $u = e^{Nt} - 1$. Substituting these values in (b) and solving for C

$$C = -2 \log_e 2.$$

Substituting this value in (b) and introducing the values of $(u + 1)$ and $(u + 2)$

$$s = \frac{2 v'}{N} \log_e (e^{Nt} + 1) - \frac{v'}{N} \log_e e^{Nt} - \frac{v'}{N} 2 \log_e 2 \quad \ldots\ldots\ldots \quad (c)$$

To the second number of (c) adding and subtracting $\frac{2 v'}{N} \log_e e^{Nt}$,

and collecting similar terms

$$s = v' \left[\frac{2}{N} (\log_e (e^{Nt} + 1) - \log_e e^{Nt}) + \left(\frac{2}{N} - \frac{1}{N} \right) \log. e^{Nt} - \frac{2}{N} \log_e 2 \right]$$

Since $\log_e e^{Nt} = Nt$, and

$$\log_e (e^{Nt} + 1) - \log_e e^{Nt} = \log_e \frac{e^{Nt} + 1}{e^{Nt}} = \log_e \left(1 + \frac{1}{e^{Nt}} \right)$$

there results finally

$$s = v' \left[t - \frac{2}{N} \log_e 2 + \frac{2}{N} \log_e \left(1 + \frac{1}{e^{Nt}} \right) \right]$$

Compare Rühlmann's "Hydromechanik," p. 699, for a similar solution.

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To determine the length of the period of time required before the velocity of particles of different sizes becomes practically uniform, M. Vauthier has expanded equation (6) by division into

$$v = v' \left[1 - \frac{2}{e^{Nt}} + \frac{2}{e^{2Nt}} - \frac{2}{e^{3Nt}} + \ldots\ldots\ldots\ldots \right]$$

For all but very large bodies all but the first two terms may be neglected. To find the length of time before the actual velocity will differ from the velocity limit by $\frac{1}{1000}$, one may put

$$\frac{2}{e^{Nt}} = \frac{1}{1000},$$

where $e^{Nt} = 2\,000$, and

$$t = \frac{\log.\ 2\,000}{N \log.\ e} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

From equations (5) and (11) M. Vauthier has prepared a table for particles of different diameters, showing the length of the path described in the first four seconds, and the length of time elapsing before the actual velocity lacks only $\frac{1}{1000}$ of the velocity limit v'.

This table shows how rapidly the velocity of fall approaches the velocity limit in each case, especially with the smaller particles. It is not until the diameter of the particle becomes as great as 1 m. that there is an appreciable difference between the two at the end of the third second, and even then the difference is slight.

Assuming a particle free to descend with a vertical velocity v in a current of water whose mean horizontal velocity is u, its vertical velocity, except in the case of very large bodies, may be put equal to the corresponding velocity limit v'.

The direction of its path, while not a straight line because of the relative velocities of the filaments of water, will yet, in general, form an angle with the horizon whose tangent is $\frac{v'}{u}$.

If the height above the bottom at starting was z it will reach the bed of the stream after a time

$$t = \frac{z}{v'}$$

and at a distance down stream

$$l = z \frac{v'}{u}.$$

With numerical values M. Vauthier obtains the following results:

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Diameter of particle $2r$ (in meters).	Velocity of current u (in meters).	Original height above bed z (in meters).	Time in sinking to bottom t (in seconds).	Distance traversed down stream l (in meters).
0.0001 (mud)	1.00	1.00	19.38	19.38
0.001 (sand)	1.00	1.00	6.12	6.12
0.01 (gravel)	1.00	1.00	1.94	1.94

Suppose this body, falling through the water with a vertical velocity v', meets an upward accidental current with a vertical component equal to v'. It would be kept in suspension so long as the current endured. This is held by M. Vauthier to explain the phenomenon of suspension.

He draws the following conclusions from his study:

"(*a*.) Water does not possess a special property by virtue of which it holds in suspension minute particles of a density superior to its own.

"(*b*.) These particles always move toward the bottom with a velocity which depends upon their density and which is inversely as the square root of their transversal dimensions.

"(*c*.) From the value of these velocities, for materials of a density similar to those which form the surfaces of the beds of water courses, the effects of displacement and of transport observed in streams and rivers is very well explained by the single fact of accidental or permanent currents which act upon the bottom."

In his "Hydraulique"[1] M. Flamant has brought together the most valuable parts of the theories advanced by Dupuit, Vauthier, Partiot, Sainjon and Lechalas. His work is of especial interest in that he calls attention to the bearing upon this question of an article by M. Du Boys[2] intended to complete Dupuit's explanation of the increased velocity of a surface float over that of the mean of the surrounding filaments of water. M. Du Boys has completed the explanation of Dupuit by adding that, while the displaced water and the floating body are alike subjected to the accelerating force due to gravity, yet the resistances to which they are subjected are different. In the displaced water, a portion of the gravity work is lost from the non-parallelism of the filaments and the consequent internal frictions, while in the floating body all the accelerating force due to gravity is used in overcoming the friction on the sides and in producing the increased velocity.

[1] "Mécanique Appliquée, Hydraulique" pp. 290–311. M. A. Flamant, Paris, 1891. Baudry & Cie.

[2] *Annales des Ponts et Chaussées*, 1886, I, p. 199.

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In summing up the results of his study, M. Flamant expresses the belief that the power of suspension increases with the quantity $\frac{d\,v}{d\,z}$, with the mean or bottom velocity and with the depth.

Experiments reported by Mr. G. F. Deacon in connection with studies for the Manchester Ship Canal give an accurate description of the detailed action of flowing water upon a bed of sand. His summary of results[1] will be reproduced here.

"The observations were made in a long flat-bottomed trough with glass sides by means of which the behavior of the sand could be accurately observed. The sand was from the estuary of the Mersey, the quantities moved were weighed and the surface velocities of the water carefully measured. When water flowed with a steadily increasing velocity over a surface of such sand, fine pieces of broken shell were first moved, and the surface velocity required to produce such movements was considerably less than 1 ft. per second. At such velocities, however, the sand proper was perfectly stable, and however long the flow continued it remained undisturbed; but the fine pieces of shells at the surface of the sand moved in spasmodic leaps, accumulating wherever the velocity was somewhat less.

"The first movement of sand began at a surface velocity of 1.3 ft. per second. This movement was confined to the smaller isolated grains; and if the same velocity was maintained, the grains so moved ranged themselves in parallel bands perpendicular to the direction of the current, each band taking the form of the well-known sand ripples of the sea shore or sand-bottomed stream, with its flat slope upwards and its steep slope downwards in the direction of the current. At this velocity the profile of each sand ripple had a very slow motion of translation, caused by particles running up the flatter slope and toppling over the crest. The steep downward slope was, therefore, being constantly advanced at the expense of the denudation of the less steep upward slope. At a surface velocity of 1.5 ft. per second the sand ripples were very perfect and traveled with the stream at a speed of about $\frac{1}{2100}$ of the surface velocity. At a surface velocity of 1.75, the ratio was reduced to about $\frac{1}{1050}$, and at a surface velocity of 2 ft. to $\frac{1}{480}$. A critical velocity was reached when the surface of the water moved at 2.125 ft. per second, when the sand ripples became very irregular, indicating greatly increased unsteadiness of motion of the water. Up to this point the whole amount of scour was represented by the volume of the sand waves multiplied by an exceedingly low velocity, always less than the $\frac{1}{480}$ part of the surface velocity of the water. At about this critical velocity of 2.1 ft. per second, the particles rolled by the water up the flat slope, instead of toppling over the steep

[1] *Proceedings* of the Institution of Civil Engineers, 1894, Vol. 118, pp. 93–95.

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slope, were occasionally carried by the water direct to the next crest; and as the velocity of the water was gradually increased, an increasing bombardment of each crest by the crest behind it took place.

"At about 2.5 ft. per second, another critical velocity was reached and many of the little projectiles cleared the top of the first or even of the second crest ahead of that from which they were fired. At surface velocities of 2.6 to 2.8 ft. per second, the sand ripples became more and more ghost-like, until, at 2.9 ft. per second, they were wholly merged in particles of sand rushing along with the water in suspension. After this the scour was of a totally different character; the sand and water became mixed, and a constant process of lifting, carrying and depositing of individual particles ensued, the sand being stirred to a depth and lifted to a height dependent upon the velocity."

Mr. Deacon refers to the theory that the weight of sand moved is proportional to the sixth power of the velocity of the water and believes the method of determination of this law to be fallacious. His observations showed that, within the limits of the experiments, the weight of material transported was proportional to the fifth power of the surface velocity or possibly a little more. Two curves are given expressing the results. One shows the relation between the surface velocity and the solid discharge in pounds of sand; the other, the ratio between the surface velocity of the current and the velocity of translation of the crests of the sand ripples.

M. Gallois has described[1] a method of experiment,[2] which throws light upon the problem of suspension. A glass bottle 3 ins. in diameter is used and its flat bottom covered to a depth of 0.2 in. with clean sand. By corking so as to exclude all air and rotating rapidly by means of a twisted cord or a turn-table, the sand is thrown by centrifugal force against the sides of the bottle. The motion of the bottle is communicated to the water progressively from the sides to the center, the sand remaining at the outside. If the bottle be suddenly stopped when the velocity of the water has come to equal its own, the sand will at once project itself from the sides to the center in a cloud, gradually subsiding to form a cone at the bottom, with a vertical axis, whose length increases with the velocity of rotation.

This cone flattens with decrease in velocity, until in still water it assumes the corresponding slope of equilibrium for sand. M. Gallois

[1] *Le Genie Civil.* See *Engineering News*, March 23, 1893, for a brief digest.

[2] Suggested by Dupuit in 1848. See page 360, and also "Etudes sur le Mouvement des Eaux," Dupuit, pp. 216-217; Flamant "Hydraulique," 1891, p. 302. Footnote.

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explains this phenomenon as follows: When the rotation of the bottle is stopped, the water continues to revolve, but is gradually brought to rest by the friction from the sides of the bottle. Since this retarding force communicates its action progressively from the outside inward, the interior filaments soon attain a relative velocity with reference to the outer ones which increases toward the axis of rotation. The sand is pushed toward the center with a force which is proportional to the velocity of the fluid. Consequently the cone flattens as the velocity of rotation decreases.

M. Fargue has recently described[1] some experiments of a similar nature, started by him in 1872 and repeated lately at Rouen and Langon. The apparatus used consisted of a circular disc upon which a zinc annular ring, about 0.30 m. high, was fixed. The internal and external radii were respectively 0.50 m. and 1.0 m. The disc was so mounted upon a vertical axis as to be given any desired rate of rotation.

By partly filling the ring with water and carefully increasing the rate of rotation a paraboloid of revolution was soon formed by the water surface.

If a uniform bed of sand and gravel was placed on the bottom before rotation began, and a number of floats at the surface, certain phenomena were seen to occur.

Up to a velocity of 1 rotation in 4 seconds the solids remained unmoved on the bottom. When the time of revolution had decreased to 3.5 seconds, isolated grains of sand and gravel moved to the concave side. This radial movement increased with the speed until, at a velocity of 1 turn in 2½ seconds, the entire mass of gravel was collected on the concave side and showed a somewhat regular surface. The floats, on the other hand, gradually descended the surface of the paraboloid until certain ones became stranded on the convex side or the bottom.

After the conditions had become fixed, the disc was suddenly stopped. The water continued its motion in a state of agitation corresponding to the angular velocity at the moment of arrest. The hollow formed toward the axis was at once filled and the surface became horizontal. The gravel was carried toward the center, with a rapidity corresponding to the angular velocity at which the disc was stopped.

[1] "Expériences relatives à l'Action de l'Eau Courante sur un Fond de Sable." Paris M. Fargue, Inspecteur-Général des Ponts et Chaussées, *Annales des Ponts et Chaussées*, March, 1894.

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When this velocity was 0.74 (time of rotation, 8.5 seconds), the materials covered the bottom almost uniformly. The disc was only bare for a discontinuous strip at the outer edge. When the velocity of stoppage was 1.11 (time of rotation, 5¾ seconds), the sand and fine gravel moved rapidly to the convex wall and the average gravel spread itself almost uniformly, except that only a few of the large particles remained at the concave wall. When it was 3.14 (time, 2 seconds), all the gravel was violently thrown toward the center and the fine sands followed in spirals of varying lengths. There was little regularity in the motion of the floats, though they generally kept to the concave bank.

PART II.—DISCUSSION OF OBSERVED DATA.

The extent of the erosive action of water courses marks it as the greatest factor in that definite movement of the materials of the earth's surface from the high toward the low latitudes, which the modern "Doctrine of Isostacy" has sought to explain by a reverse movement underneath and a subsequent elevation[1].

The study of the torrents of Switzerland and Italy suffices to show the size of individual blocks which may be moved along the bed of a stream[2] or even carried freely in suspension. The burden of detritus brought down in the middle and side moraines of the Unteraargletscher in the Bernese Oberland is spread over a wide area by the headwaters of the Aar River, forming a waste of heavy boulders and coarse gravel covering ½ square mile. Through this wilderness of stone, the milky waters of the river find a tortuous path, carrying in suspension to the Lake of Brienz below the particles of powdered rock ground from the sides of the valley by the daily motion[3] of the glacier. It is the presence of this so-called "gletschermilch," which gives to the Swiss lakes a part of their peculiar and beautiful coloring.

These short but destructive torrents divide themselves naturally into a "sammelgebiet or erosionsgebiet," where the water and solid material is gathered, the "gebiet des murgangs"[4] forming a canal

[1] Compare "Theory of the Earth's Rotation and its Interior Heat," pp. 26–32, Elon Huntington. Rochester, N. Y., 1895.

[2] For a graphic description of the descent of material in a mountain torrent, see Lechalas, "Hydraulique Fluviale," Annexes, pp. 424–428.

[3] The Unteraargletscher has a velocity down the valley of 0.50 m. per day. It was here that Agassiz made his glacier measurements. The Rhône Glacier, separated from the valley of the Aar only by the Nägelisgrätli divide, has a daily velocity of 1.0 m.

[4] For examples of dangerous "murgänge," read Riedel's "Ueber Geschiebe Führung und Murgänge der Wildbäche." *Zeitschrift des Oesterrich-Ingen- und Arch.- Vereins*, 1871, pp. 113 and 151.

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through which the semi-fluid mass passes at considerable velocity and with little deposit, the "ablagerungsgebiet," where the solid material is deposited in the main valley, forming a clearly defined cone, with its apex at the point where the torrent issues from the mountain. Lastly, the "ablauf"[1] or bed through which the water, relieved of the mass of its burden, finds its way to the main water course.

A photograph, taken during the summer of 1895, shows clearly these lines of demarcation in the two torrents close above Guttannen on the west side of the lower Haslithal. The axis of each is approximately at right angles to the Aar, into which they discharge. At the foot of this same valley on the eastern slope, above Brienz, lies the small Swiss village of Neuschwanden. At its edge, through an abrupt chasm in the mountain side, and so close as to render the danger to the village an imminent one, issues the cone of dejection of the Lammbach, probably the most destructive in Switzerland. The huge mass of stone and boulders covers a fan-shaped area of approximately a square mile and is largely devoid of vegetation. The slope is nearly uniform from the apex to the banks of the Aar, which it has forced against the further side of the Haslithal. An approximate measurement, made by the author in August, 1895, showed this slope to be about 8 degrees. At that time the side toward Neuschwanden was overlaid with the fresh "murgang" or lava-like mass of gravel and boulders of the preceding autumn which had formed a semicircular cordon about the village and was only deflected from the houses by heavy guide walls. The upper surface is nearly plane and the stream does not, as might be expected, thin out gradually to the edges. It forms a bed of nearly uniform thickness, forking into various divisions at the apex of the cone, while each edge is sharp and clearly defined, marking an abrupt descent to the bottom. In general one may liken the form of these streams to that of those beds of broken stone carefully arranged in prismoidal form one sees in American cities.

That something of an analogous nature takes place in all larger water courses is certain. The difference is one of degree and not of kind. The variance of opinion among authorities now hinges on the ratio between the total amount so moving, in a given river, and the amount carried in intermittent or permanent suspension.

[1] The French writers use only the first three divisions and the corresponding terms "bassin de réception," "canal d'ecoulement" and "cône de déjection."—See Surell "Etude sur les Torrents des Hautes-Alpes." Paris, 1870–72, Dunod.

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Piles driven up stream from a caisson of the St. Charles Bridge over the Missouri are said to have been found under the caisson when it reached bed rock.[1] Jas. B. Eads describes the sand on the bed of the Mississippi at the St. Louis Bridge as moving for at least 3 ft. in depth, with a velocity decreasing below the surface.[2] A pile embedded upright in the sand has been seen to move bodily down stream.[3] The velocity of movement in this sense has been determined by means of stakes driven in the bed of the Loire.[4] Other recorded cases are numerous and need not be multiplied here.

The transportation of coarse gravel in free suspension is but another order of the same phenomenon. It has been observed in the Garonne, when dikes have been broken through, and gravel, borne in the upper laminæ of the current, has been carried over the breach and deposited in the fields beyond.[5] In a similar case, masses of gravel were carried over a dike below Pittsburgh[6] and deposited down stream, filling up hollows which had previously existed there.

The law of decrease in mean velocity from the rise to the embouchure of rivers is closely followed by the steady decrease in size of the particles forming its bed and strewn along its banks. The ratio between the amount of solid matter entrained and that of the liquid at any point in a stream has been called by M. Fargue its torrential coefficient. This should decrease from source to mouth with the fall and the mean velocity. M. E. Charlon[7] has made use of the law in the deduction of a formula, by means of which he computes the velocity of a stream from the size of the materials transported by it. The question of corresponding decrease in amount of suspended matter, per cubic foot of water, is a disputed one. M. Fargue[8] holds the view that rivers become more and more muddy toward their embouchures, due to the accumulated transformation of the coarse into fine materials by friction. M. Partiot,[9] on the contrary, states that 300 measure-

[1] "The Mississippi as a Silt Bearer." R. E. McMath. *Van Nostrand's Engineering Magazine*, Vol. XX, 1879, p. 227.

[2] "Report of Chief Engineer of St. Louis Bridge." J. B. Eads, June, 1868, p. 21. Quoted by R. E. McMath in above paper.

[3] *Engineering News*, Feb. 9, 1884, p. 65.

[4] See Partiot. "Les Sables de la Loire," p. 43.

[5] The same, p. 23.

[6] "Report of Chief of Engineers, United States Army," 1876. II, p. 5.

[7] See *Le Génie Civil*, Vol. XVII, 1890, p. 170. Note giving formula in *Proceedings* of the Institution of Civil Engineers, Vol. 102, p. 350.

[8] "Etude sur la Largeur du Lit Moyen de la Garonne," pp. 12, 13. M. Fargue. *Annales des Ponts et Chaussées*, October, 1882.

[9] "Mémoire sur les Sables de la Loire." M. Partiot, p. 21.

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ments in the Loire, during the floods of 1856, showed the turbidity to decrease regularly toward the sea. Measurements continued throughout the flood showed, proceeding down stream, the weights of sediment per cubic meter of water to be—

At Feurs ....300 grs.	At Nevers ...210 grs.	At Tours.....212 grs.
Roanne...242 "	Gien .....223 "	Saumur...177 "
Digouin ..191 "	Orléans ..237 "	Nantes....150 "

M. Partiot explains the anomalies shown between Nevers and Orléans by the entry, between those points, of tributaries heavily charged with silt.

A comparison of various measurements in the Mississippi was undertaken by the author. The results would seem to bear out M. Partiot's view.

The available data are the Carrollton and Columbus measurements of 1851 and 1858, the measurements of 1879, at Helena and at St. Louis.[1] The two former were taken during floods. The two latter at a medium stage. The Columbus measurements represent only surface specimens. The Carrollton samples were taken with a defective apparatus. These facts render it impossible to draw any conclusions from a comparison of the 1851 and 1858 results with those of 1879, which seem more reliable.

However that may be, a mean of the results at Columbus and at Carrollton, from the second week of March to the second week of November of their respective years (1851, 1858), shows the proportion of sediment to water, by weight, to be—

Columbus	.000749
Carrollton	.000601

an evident decrease in sediment at the lower station.[2]

There is an apparent co-ordination between the two 1879 measurements which gives more weight to the results obtained. Taking the mean only of the top and bottom measurements, as no mid-depth quantities were taken at Helena, and covering the period from April

[1] See a valuable article by R. E. McMath, in *Van Nostrand's Engineering Magazine*, Vol. 28, 1883, p. 33.

[2] During these measurements the mean velocity of the river ranged at Carrollton from 6 to 1.7 ft. per second, and at Columbus from 8 to 1.5 ft. per second, being, as a whole, considerably higher at Columbus.

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10th to June 18th, 1879, at Helena, and April 14th to June 25th, at St. Louis, the proportions of sediment to water, by weight, are—

St. Louis	.002046
Helena	.001079

a much greater decrease toward the river mouth.[1]

The law cannot be demonstrated from the measurements now available. That the effect of interaction among the solids moved on the bed should show a cumulative effect in the increased number of fine suspended particles down stream is to be expected, and is shown in Nature by the increased fineness of the deposits. That it should manifest itself in an increased weight of suspended matter per cubic foot, as held by M. Fargue,[2] does not appear to be substantiated by the limited number of observations at hand.

The question is: Will a stream moving at a given velocity sustain a greater weight of fine particles per cubic unit of water than of large ones of the same density? If so, then a heavier load per cubic foot may be carried at the embouchure of rivers with the same expenditure of energy than in their higher reaches.

Mr. G. K. Gilbert[3] has shown that the same consumption of energy will hold in suspension a greater load of fine than of coarse material of like density.

This may be shown as follows: Assume a stretch in the lower course of a river bounded by the cross-sections *A* and *B*. Assume the kinetic energy at the two points to be the same, so that the whole gravity work done by the weight of the stream in its descent is used up in external and internal frictional resistance. Suppose an inch cube of stone introduced at the surface at *A*. The total energy of the stream has now been increased. The cube reaches the bottom at *B*. It can only act on the bed between the two points by pressure, but as the friction on the bottom is independent of fluid pressure, this friction is not increased. If the cube sinks at the same rate it would have chosen in quiescent water, it makes no demand upon the energy of the

[1] At Helena, the mean velocity ranged from 4.26 to 3.23 ft. per second, while at St. Louis it varied from 7.21 to 4.0 ft. per second, averaging considerably higher than at Helena.

[2] M. Fargue says ("La Largeur du Lit Moyen de la Garonne," p. 13): "Il s'opère donc, de l'amont vers d'aval, une transformation dans la qualité et dans le mode de transport du débit solide: le débit en gros matériaux traînés sur le fond, qui n'a lieu que sous l'influence de vitesses notables, va en diminuant; celui des matériaux ténus, en suspension dans l'eau et obéissant aux faibles vitesses, va au contraire en augmentant. * * * les eaux deviennent en effet de plus en plus vaseuses à mesure qu'on se rapproche de la mer."

[3] *American Journal of Science*, July and August, 1876, Part II; also abstract in *Engineering News*, August 19th, 1876.

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stream. If it sinks more slowly, the difference between the distance actually sunk and the distance which would have been covered in quiet water during the time of transit from A to B, multiplied by the weight of the cube in water, measures the demand upon the stream's energy.

Suppose the same stone to be pulverized into small cubes and again introduced at A. The weight in water is unchanged. The draft upon the stream's energy between A and B is computed in the same way as before and found to be less, because the difference between the distance sunk in quiet water and in the actual case is less. The reason why the small particles sink more slowly is because the collective area at right angles to the motion is greater, and so requires a smaller value of the velocity of sinking in order to keep the total resistance to motion,

$$P = k\gamma F\frac{v^2}{2g}$$

a constant in the two cases. This is required because the work done by the cube in sinking to the bottom must be the same as that done by its component parts in covering the same distance.

It is shown, then, that a less consumption of the stream's energy between A and B is required to suspend the same weight of small particles than where the grains are of the same density and larger size. It follows that the same expenditure of energy will suspend a greater weight of the small particles per cubic foot of water.

This may offer an explanation of the phenomenon observed at Columbus and advanced by General Abbot[1] to prove that no relation exists between velocity and weight of sediment per cubic foot of water. He states that the Ohio and Missouri Rivers move side by side at Columbus in the bed of the Mississippi without mingling their waters, and that, while their velocity is common, the Ohio water has only three-fourths as much sediment per cubic unit as the Missouri water.

If they are actuated by the same velocity it may be assumed that there is the same amount of energy per cubic foot in each case diverted to the suspension of sediment. The sediment of the Missouri is much more comminuted than that of the Ohio, as shown by the appearance of the two streams. An excess in weight of sediment per cubic foot is to be expected, then, in the Missouri water, as the measurements showed, even though the velocity is the same in both.

[1] *Van Nostrand's Engineering Magazine*, Vol. XX, 1879, p. 3.

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The problem now becomes one dependent upon the circumstances of each case. If the energy available for the work of suspension per cubic unit of water is the same at the mouth as at the head waters of a stream, the weight of sediment per cubic unit will be greater. This excess will diminish and finally become negative as the ratio of the available energy at the mouth and head waters becomes less.

In general, it would seem that the available energy should decrease rapidly toward the embouchure and be accompanied by some slight decrease in the weight of sediment carried per cubic foot of water.

1. Minor Agents Influencing Sedimentation.

Temperature.—Chemical precipitation, in general, takes place more easily at higher temperatures. The same law appears to obtain in the case of matter in mechanical suspension. The author's attention was first directed to the question by Mr. Allen Hazen, of Boston, who had noticed an appreciable increase in deposition of suspended matter at higher temperatures in the sewage at the Lawrence, Mass., Experiment Station. Bouniceau[1] and Partiot[2] are agreed that river deposits are greater in summer than in winter. The sediment observations of Prof. Riddell[3], and those of Prof. Forshey[4], at Carrollton, on Mississippi water both give corresponding temperatures. The former lasted from May 21st to August 13th, the temperature gradually rising from 72° to 84° Fahr. The corresponding amounts of suspended matter show an irregular but still perceptible decrease. They follow, however, much more closely the fluctuations in the river surface above low water, so that the element of decreased depth and consequent decrease in velocity of flow enters in as a more potent factor in producing the same result.

The sediment curves at both Columbus and Carrollton[5] seem to show an increase in suspended matter during the summer months of June, July, August and September, the temperature of the river water at Carrollton reaching a maximum of 86° Fahr. in August and descending very regularly to a minimum of 39° in February.

[1] "Etude sur la navigation des rivières à marées." M. Bouniceau, 1845. Quoted in *Proceedings* of the Institution of Civil Engineers, Vol. 66, p. 5.

[2] "Mémoire sur les Sables de la Loire," p. 22; M. Partiot. *Annales des Ponts et Chaussées*, I, 1871.

[3] "Report to American Association of Geologists and Naturalists, 1846. Quoted by Humphreys and Abbot, "Report on Mississippi River," p. 142.

[4] "Report on Mississippi River," Humphreys and Abbot, pp. 134, 148.

[5] The same, Plates XII and XIII.

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It is evident that river observations are little fitted for the study of this question because of the complexity of the elements involved in fluvial motion. A simple laboratory experiment on the length of time acquired for mechanical precipitation in quiescent water under different temperatures would determine the matter. The yearly range in temperature in rivers is not great, and its influence on sedimentation will be very limited. In the case of flow in sewers it may assume more of practical importance.

A large number of different measurements in the Elbe[1], made under various conditions, failed to show any change in temperature with depth. In the Mississippi River the difference between surface and bottom temperatures is usually too small to be registered by an ordinary thermometer. The maximum difference is a small fraction of a degree.[2]

Herr Blohm[1] has assigned an important place among the causes of suspension of finer particles to a system of circulation set up by the differences in temperature. He calls attention to the fact that water reaches its greatest density at 3.5° R. (39° Fahr.) and that laminæ at a less temperature than this would tend to rise to the surface, as well as those at higher temperatures. The result would be a mixture tending to produce the uniformity actually observed at all depths. The tendency of the warmer laminæ to rise would be equal, in his opinion, to the tendency of the finer particles of sediment to sink in obedience to gravity.

The Mississippi may be taken as an index of the rivers of the temperate zones. Its waters at Carrollton during two years' observations never exceeded the temperature of maximum density, so that, in this case at least, there could have been no circulation, from this cause, of colder water from below to the surface as an equalizer of temperatures.

Light.—The slight molecular agitation caused by the penetration of light has been shown to be sufficient to affect the rate of sedimentation in quiescent water. Mr. Andrew Brown[3] found that a phial of turbid water had a uniform tendency to deposit its sediment most

[1] "Ueber die in fliessenden Wasser suspendirt enthaltenen Sinkstoffe," Blohm. *Zeitschrift des Architekten und Ingenieur-Vereins*, Hanover, 1867, pp. 277–278.

[2] Lieutenant Marr in "Report on Mississippi River," Humphreys and Abbot, 1876, p. 149.

[3] *Proceedings* American Association for the Advancement of Science, 1848; also, Humphreys and Abbot's "Report on the Mississippi," 1876, p. 144.

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rapidly in the portions protected from the light, the surface of the deposit showing a corresponding inclination.

Viscosity.—At low water, under a hot sun, M. Partiot[1] has seen the rising tide, at the embouchure of the Loire, float off patches of sand from the bars and carry them upon its surface so long as it remained undisturbed by waves. A similar phenomenon is noted in the *American Journal of Science*, December, 1890.[2] Blotches of sand, 1 in. in diameter at first, which later joined themselves into 6-in. squares, were eroded from a bank forming an angle of 150° with the water surface. These were seen floating on the surface half a mile down stream, and, if disturbed, would rapidly sink to the bottom. An oiled needle will float on the surface of water if carefully placed in position. Phenomena of this nature are due to what may be called superficial viscosity, which has a greater intensity than the viscosity in the interior of the fluid.

That this latter influence has a part in the suspension of sediment is shown by the length of time required for quiescent turbid water to clear itself. Experiments showed turbidity in water taken from the Garonne after eight days and muddy water from the Elbe made no perceptible deposit until after a lapse of 24 hours.[3] In water taken from the Mississippi at St. Louis in 1865, Mr. Flad[4] found that for a total of 1 000 parts in suspension at the beginning of the experiment,

944.50	parts	had	settled	during	the	first	24	hours.
22.35	"	"	"	"	"	second	24	"
2.92	"	"	"	"	"	"	48	"
30.23	"	were	still[5]	in	suspension	after	96	"

Changes in viscosity and consequent suspending power are a probable concomitant of changes in temperature.

Salt Water.—Observations made in 1839 by Sidell[6] at the mouths of the Mississippi showed that the river water alone required from 10 to 14 days to settle. The admixture of salt in any form reduced the time of settling to between 14 and 18 hours. Mr. Gould[7] found that a few

[1] "Sables de la Loire," p. 36.
[2] Noted in *Engineering Record*, December 27, 1890, p. 65.
[3] Blohm in *Zeitschrift des Architekten- und Ingenieur- Vereins*, Hanover, 1867, p. 245.
[4] "Silt Movement by the Mississippi," R. E. McMath. *Van Nostrand's Engineering Magazine*, 1883, p. 33.
[5] Since the present article went to press the author's attention has been called to an article by Mr. Carl Barus (Bulletin No. 36 of the U. S. Geological Survey), in which he shows that a degree of comminution can be secured such that deposition will never take place.
[6] "Report on the Mississippi River," 1876. Appendix A, p. 500.
[7] "Report of Chief of Engineers, United States Army," 1875, II, p. 36.

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pinches of salt thrown into a tumbler containing muddy water from the bottom of the Savannah River caused a much more rapid deposit. Mr. Fargue[1] has found that the same amount of mud introduced into a glass of fresh water and into a glass of salt water shows a difference in the period of settling. The salt water is clear after six hours of repose. To attain the same result requires eighteen hours in the fresh water.

This property of saline solutions has an important bearing on the formation of bars at mouths of rivers discharging into salt seas. The load of detritus will be dropped sooner than if the receiving body were fresh water.

Action of Waves.—The formation of bars in deep water at the mouths of tidal estuaries has in late years come to be attributed to wave action upon the detritus discharged by the river rather than to the simple process of deposition itself. It is the dynamic effect of the waves which heaps up the bars.[2] All sea beaches show this action so clearly that there can be little doubt as to its influence, in a lesser degree, on the movement of detritus in rivers. Observation has shown that sands are often moved when the bottom velocity is such as to be an insufficient cause. Mr. P. O'Meara[3] has observed this motion by diving to the bed of a tidal channel where the bottom velocity was too slight, unaided, to move the sands. He found that the sand at and near the bottom, under a depth of 10 ft., had an oscillatory motion corresponding to the 6 and 8-in. waves passing above. At the center of the wave passage the sand reached a considerable velocity; at its end the motion ceased and even seemed to be reversed. He holds that this action may be perceptible to depths of 40 or 50 ft. Waves of translation stir the water to an infinite depth, theoretically, their velocity of translation being dependent upon the depth. Such waves will be confined to tidal estuaries. Waves of oscillation, such as the wind ripples in rivers, are felt, however, to considerable depths.

At Cherbourg[4] these waves cease to act on the piers at a depth of

[1] "Étude sur la Largeur du Lit Moyen de la Garonne." *Annales des Ponts et Chaussées*, Oct., 1882, p. 21 (footnote).

[2] A formula for the scouring power of waves, giving relation between height of wave and size of particle moved, is developed by Mr. W. Smith in *Proceedings* of the Institution of Civil Engineers, Vol. 100, p. 201.

[3] *Proceedings* of the Institution of Civil Engineers, Vol. 118, pp. 84, 85. Noted, also, in *Engineering Record*, Feb. 23, 1895, p. 219.

[4] "Les Marées Fluviales," M. Comoy, 1881, p. 23.

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about 25 ft. At Algiers the limit is 35 ft. There the sands cease to be moved at depths between 50 and 100 ft., while the limit for muds is 450 ft. A visit to the harbor of Algiers during a heavy blow showed its peculiarly exposed position so that these figures may be reasonably considered maxima.

Action of Ice.—The removal to great distances of boulders which neither ordinary nor flood velocities could move has been attributed to the transporting power of ice. There is a tendency in rivers to form what is called anchor ice at the bed and sides when the water is shallow. This ice attaches itself to the solids in its vicinity, and, because of its slight specific gravity, is easily detached by flood velocities and carried with its load down stream. M. Partiot[1] calls attention to this action on sand shoals barely covered by water from which the surface layer is detached by the floating away of the ice.

Action of Sediment in Diminishing Velocity.—Mr. Baldwin Latham reports[2] observations covering a series of years which seem to show that the velocity of turbid water for the same depth and fall is less than that of clear water. He holds that this difference bears a ratio to the amount of turbidity, and is caused by the work used in transporting the material. The discharge of clear water multiplied by its velocity corresponded closely to the combined weight of sediment and water in the corresponding case, multiplied by its mean velocity. Mr. G. K. Gilbert has reached the same conclusion.[3] He states that the total energy of a clear stream is used up in friction on its bottom; that this friction is directly proportional to its velocity. When detritus is carried a certain amount of the energy of the stream is used to keep it in suspension, and this takes place at the expense of friction and consequently of velocity. It is to be remembered, however, that the total energy of the stream has, in the meantime, been increased by the addition of the energy represented by the vertical fall of the solid particles.

The law of the conservation of energy will not admit of any other

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[1] "Les Sables de la Loire," p. 36.

[2] *Proceedings* of the Institute of Civil Engineers, Vol. 71, p. 46.

[3] "The Colorado Plateau Province as a Field for Geological Study," *American Journal of Science*, July and August, 1876. Part II. Abstracted in *Engineering News*, August 19th, 1876.

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decision in this matter, though the statement has sometimes been made that such a retardation of velocity does not exist[1].

Mr. Gilbert's statement that the work done by a clear stream is entirely used up in friction on the bed is somewhat at variance with the attitude of the best science of the present day.[2] The energy consumed by intermolecular resistances caused by the complex motion in the interior of the liquid is much greater than that actually used at the earth and air profiles. It should be added, however, that these intricate movements are induced by the bed's rugosities. In general, it may be said, that the total energy is used in friction, through which it is transformed into heat energy.

Assume a portion of a clear stream between the sections A and B. Suppose no difference of kinetic energy between the two stations, then the total energy of the stream expended is used in work done on friction. Introduce a mass of sediment in suspension at A and a demand

[1] See "Silt Movement by the Mississippi," R. E. McMath. *Van Nostrand's Engineering Magazine*, 1883, p. 36. Mr. McMath says: "We have seen that transportation of silt (up to the point of impaired fluidity) is not at the expense of the stream's motion. The work of erosion and suspension is done by the stream, whose velocity must be diminished compared with flow under a like head in a smooth channel, but if the now-yielding bed should suddenly become rigid, the same, or even greater force would be expended upon the obstructing roughness. Therefore though suspension consumes a part of the stream's force the velocity is not necessarily lessened beyond what it would be in the only alternative condition that can be considered, a rigid bed equally rough." This line of reasoning would seem to hold, so far as the actual work done upon the bed of the river between any two points is concerned. The work which would have been expended upon a rigid bed equally rough is now in part expended upon the mobile bed in the same way as before, while the residue is free to be used in carrying into suspension whatever is eroded from the bed between the sections considered. But this theory fails to take account of those external forces of Nature which are continually wearing away cliffs, disintegrating hillsides and introducing at the surface of the stream a mass of debris to be carried, for which the stream's own mechanical action is not accountable. Gravity acts as an external force where banks cave in and throw upon the stream's energy an additional burden. The burden already in suspension at the entrance to the stretch considered must be carried in addition to that considered by this theory. It is to this additional burden that a consumption of energy and consequent retardation of velocity may be attributed.

[2] See Boussinesq "Theorie des Eaux Courantes," Paris, 1872, introductory chapter. M. Boussinesq has shown that neither the friction, rightly called, upon the bed nor the added internal friction due to relative velocities of parallel filaments following stream lines is sufficient to explain the transformation of the energy of the stream, in its descent, into heat energy. He shows that if the velocity at the walls were assumed to be zero so as to attribute the whole work to friction between parallel filaments, the coefficient of interior friction is so small that the central filament, in a semicircular conduit of 1 m. radius and a fall of 1 in 10000, would acquire a velocity of 187 m. per second before equilibrium was established between the accelerating force and the fluid resistances. It is, then, to the vortices that must be attributed the largest share in this transformation. They largely increase the total interior friction.

See "Journal de M. Lionville," t. XIII, 1868. Also "Theorie des Eaux Courantes," Boussinesq, pp. 2–6.

Compare, also, Prof. Unwin in "Encyclopedia Britannica," article on Hydromechanics.

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is made on the stream's energy to keep it suspended to B. The thought at once suggests itself that the total energy has, in the meantime, been increased. In answer, it may be said that the addition has also increased the friction at the bed since the formula

$$P = k\, F\, \gamma\, \frac{v^2}{2g},$$

shows this friction to be a function of the heaviness of the fluid, which in its new compound state has been increased. These two changes tend to counteract each other and to leave still an increased demand upon the energy originally used in the passage from A to B. In consequence, there will result a retardation of velocity at B accompanied by an increase of depth if the supply be constant. For particles of a uniform size and density, this decrease in velocity will increase with the weight of the load per cubic unit of water. The decrease will be less for a given weight of fine particles than for the same weight of large ones, other conditions remaining the same.

The presence of silt, then, retards velocity in two ways:

First.—It uses an amount of the stream's energy in suspending it.

Second.—It increases the heaviness of the composite fluid and so increases friction.

2. Influence of Depth on Transporting Power.

It was once believed by hydraulicians that the adhesion between a liquid and its bed was stronger than the internal cohesion of the fluid itself. It seemed the natural deduction from the decrease in velocity observed near the banks and bed. The hypothesis then took form that the particles next the banks remained stationary, while the main current flowed by in a fluid bed of its own consistency.[1] The experiments of Darcy on deteriorated pipes showed that velocity was a function of roughness, and the incorrectness of the former assumption. Even the outermost particles of the fluid substance have a motion relative to the bed. Is this velocity influenced by the depth? Increased depth means increased weight per square unit of bed, and consequently increased pressure, but experiments have shown that not only the coefficient of fluid friction, but also the friction itself, is independent of the pressure.[2]

[1] This has been shown to be true for capillary tubes by M. Duclaux, of Clermont. See *Annales de Chemie et de Physique*, 4e Série, t. XXV, 1872. For flow in streams and large pipes it appears inadmissible.
See also "Theorie des Eaux Courantes." J. Boussinesq, pp. 1-2.

[2] "Encyclopedia Britannica," Article Hydromechanics.

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In the case of a homogeneous solid sliding down an inclined plane the coefficient of sliding friction is independent of the normal pressure, but the friction itself

$$P = f N \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

is a function of both quantities, and remains unchanged at all velocities. The liquid prism, sliding down an inclined bed, acts according to other laws so far as frictional resistance is concerned. Here again,

$$P = (k\gamma)\left(F\frac{v^2}{2g}\right)$$

or

$$P = f_1 N_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

an equation of the same form as before, but made up of quantities formed in a different way. In equation (1) N represents the normal component of the body's weight, is proportional to depth when the prism is homogeneous, and is independent of velocity. In (2) N_1 again represents weight, but this weight is directly proportional to a velocity height, and is independent of depth. Increased depth, velocities remaining constant throughout, will have no effect on friction and hence produce no change in scouring action.

The case is sometimes cited, as a substantiation of the view that depth increases transporting power for the same velocity, of the increased difficulty in wading a deep stream. The example is not a good one. A man's foothold is lost sooner in this case than in a shallow ford because of his increased loss of weight rather than from an increase of transporting power, properly called.

The statement has been made[1] that in practice observation shows the scouring power of a shallow stream at a high velocity to be much less than that of a deeper river running at a slower velocity. The explanation offered by Prof. Unwin[2] would seem to account for a portion of this difference. He attributes to the deep stream the advantage that the particles of its bed may be thrown up to a greater height, and, since the velocity of descent again to the bed should be the same in both cases, will be thus carried farther down stream before being deposited.

Flood waters offer great variations of depth which may be used in the determination of comparative amounts of sediment per unit volume.

[1] See *Proceedings* of the Institution of Civil Engineers. Vol. 82, 1884, p. 31.
Mr. Law's explanation of the phenomenon is based on the incorrect assumption that fluid friction increases with pressure.

[2] "Encyclopedia Britannica," article Hydromechanics.

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These weights will vary for the same depth with the relative stage of the flood, and so complications are introduced. The earlier stages of a heavy rainfall wash down the surface particles loosened by weathering. The later portion of the storm finds a more resisting surface. M. Partiot found traces of this fact in the relative weight of sediment at different stages of the same flood. A safe basis of comparison would seem to be the same relative stages of floods of different heights when the same tributaries are discharging high water.

M. Partiot[1] has been able to detect only a slight increase in the sediment per unit volume in the Loire, with the importance of the flood. M. Fargue[2] states that the suspended sediment is very feeble at low water and reaches its maximum intensity at the flood crest.

Major Allan Cunningham[3] bases the statement that no relation exists between depth and silt intensity, upon a series of observations on the Ganges Canal.

A study of the data at hand will throw the most light on the subject. Fig. 1 gives a graphic representation of data bearing upon this point. Further curves might be added from the extensive measurements of the Mississippi Commission[4] made in 1879–81 at Carrollton, Prescott, Winona, Clayton, Hannibal and St. Louis. The Carrollton and Columbus sediment ordinates are taken from the "Report on the Mississippi" by Humphreys and Abbot, page 417—one for each week of the year. The corresponding mean gauge readings were taken from Plates XII and XIII of the same volume. The St. Louis co-ordinates are from Mr. McMath's paper in *Van Nostrand's Engineering Magazine*, 1883, page 33. The Helena co-ordinates are from the "Report of the Chief of Engineers, United States Army," 1879, Part III, page 1968. The Elbe and Maas measurements are taken from the *Zeitschrift des Architekten und Ingenieur Vereins*, Hanover, 1867, pages 290 and 291.

The method of plotting these curves must be distinctly understood. They start from no common origin. They have no quantitative relation to each other. Each represents only the general trend of direction of the number of points from which it is constructed. These are in most cases widely scattered, but their general direction is clearly defined and

[1] "Sables de la Loire," p. 22, 1871.

[2] "La Largeur du Lit Moyen de la Garonne, p. 14, 1882.

[3] See *Proceedings* of the Institution of Civil Engineers, Vol. 71, 1882, p. 35. "Roorkee Hydraulic Experiments."

[4] "Report of Chief of Engineers, U. S. Army," 1883, III, pp. 2209 and 2266.

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is fairly represented by the lines shown. It is seen at once that their direction, in each case, is such as to show an unmistakable increase in sediment per cubic foot with higher stages.[1] Each is independently constructed and all show the same thing.[2] The author can find no other values to contradict them.[3] This seems to show satisfactorily that weight of sediment per cubic foot increases as the river rises and depth increases. That it proves that transporting power in

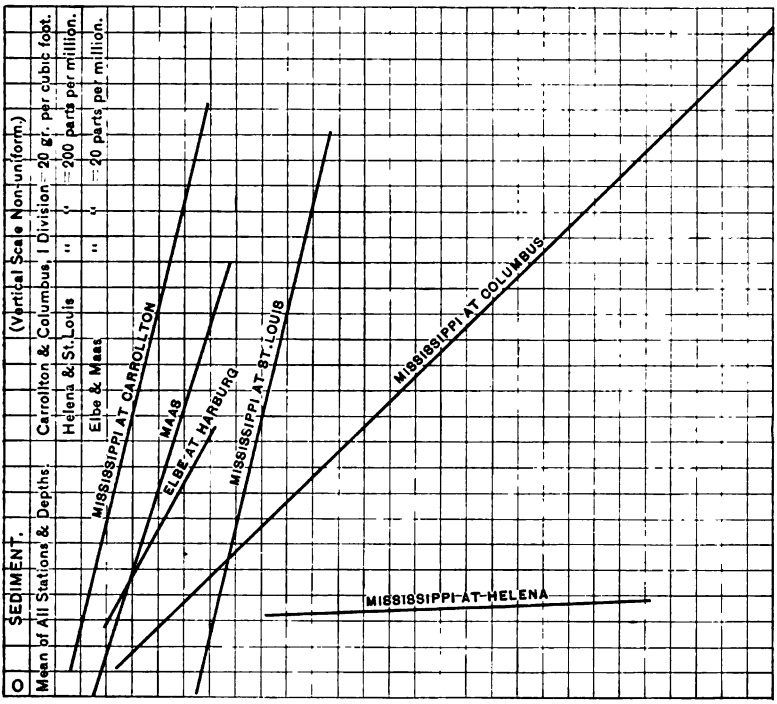

FIG. 1.

creases with depth is not claimed, for velocity increases with stage also, and either one or both together may be the cause of increased

[1] In the curves shown on Plate VIII of the "Report of the Mississippi River Commission," 1879–81 (see "Report of the Chief of Engineers, United States Army," 1883, III), this relation is only slightly traceable in the Carrollton measurements of 1879–80.

[2] Since the above curves were plotted the author has found a plate showing a series of points plotted in an analogous manner from the extensive observations of Assistant Engineer Seddon at St. Charles Mo., 1879 (see "Report of the Chief of Engineers, United States Army," 1887, Part IV, pp. 3090–96. For description of apparatus used see same report, pp. 3121–23). These points show how a curve similar to those in Fig. 1 could be drawn and are clearly confirmative of the conclusions here deduced.

[3] The results of Prof. Riddell's measurements are confirmatory. See Humphreys and Abbot's "Report on the Mississippi," 1876, p. 142.

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transport. If, however, one were to join with Humphreys and Abbot in denying any fixed connection between velocity and suspending power, then this might be looked upon as proof.

The influence of a gradual or sudden decrease in depth upon transporting power offers a range of experiment and observation which is yet to be made. There is little definitely known from measurements about the influence of such shoaling upon the curve of velocities. For unchanged width, decrease in depth means decrease in sectional area and consequent increase in mean velocity for constant discharge. Such a shoaling may be likened, in its action, to a submerged weir. When the change is sudden there will be a mass of dead water above the weir, at the bottom, forming a fluid bed upon which the discharged water flows. This would seem to indicate conditions favorable for a deposit above the obstruction, upon the same principle upon which Humphreys and Abbot explained the formation of delta bars by deposits in the dead angle caused by the fresh water flowing over the heavier salt water.

The case of movable dams in many of the continental rivers is in point. Experience, however, fails to show any shoaling of consequence above these structures.[1] The natural conclusion is that this dead water, so called, must be in a lively state of agitation, its eddies and vortices carrying up deposited material to the higher laminæ whose movement of translation carries it over the obstruction.[2]

In a gradual shoaling, with banks widening to form a pool, there is no sudden accession of vortex motion and the decrease in velocity due to enlargement of section is followed by deposits until equilibrium is established between the velocity of the stream, the resisting power of the banks and that of the bed.

The question of changes in the form of the curve of velocities, as affected by obstructions, is still in need of experimental research by measurements made at varying distances above the obstruction to determine the velocities throughout the range of the back-water.

That the friction of the air has some effect in influencing the form of this curve has been generally accepted since the Mississippi measurements were made. It is only the extent of the influence which was then claimed for this factor which has since been called in question by

[1] For substantiation of this statement see Flamant, *Annales des Ponts et Chaussées*, 1882, quoted by Lechalas, "Hydraulique Fluviale," 1884, p. 64.

[2] See "Report of the Chief of Engineers, U. S. Army," 1876, II, p. 5.

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those who have other theories for the cause of the depression of the maximum velocity below the surface.[1] That its retarding effect is less than that of the friction on the bed is usually conceded.

That both are greater than the internal friction due to laminated flow is a reasonable conclusion from the actual form of the vertical curve of velocities, substantiated by Boussinesq's demonstration[2] of the slight value of this friction of laminated flow.

Suppose any cause produces a sudden increase of roughness in the bed and so increased bottom friction. If the hypothesis of parallel filaments[3] is adopted as representing the general trend of flow, the natural conclusion is that the parabolic curve of velocities in a vertical will be tipped down stream. In other words, the lower filaments will be more retarded than the upper ones. That a gradual shoaling with a corresponding increased surface fall has the opposite effect on the curve of velocities has been shown by Dupuit[4] for the case of unchanged width.

There, by a simple numerical calculation, the bottom velocity is shown to take on a much more rapid increase than the surface velocity, so that, at the crest of the shoaling, they have become more nearly equal than before. In the contracted section below the crest the curve will have been tipped up stream, though the rapid increase of bottom friction (varying as the square of the velocity) will soon force it back to the normal position for steady flow at that velocity. He attributes to this disproportionate increase in bottom velocity with the mean velocity, the larger part of the scouring action seen immediately down stream from hydraulic constructions. This view would seem to throw additional light on the cause of non-shoaling above submerged weirs. Experiments are needed on this subject.

3. Influence of Retardation of Velocity.

What, if any, is the relation between velocity and suspension? This is a vital question in the discussion. Assume a sedimentary stream

[1] See Prof. James Thomson, "Encyclopedia Britannica," Article Hydrodynamics. Also, F. P. Stearns, *Transactions* American Society of Civil Engineers, Vol. 12, p. 331, and Vol. 7, pp. 122–130.

[2] See page 289, footnote.

[3] In spite of the statement often seen, that this in no wise corresponds to the complex motion seen in rivers, especially the Mississippi, an ordinary inspection of streams, even in times of flood, shows it to be more reasonable than any other supposition yet advanced, and to represent, in an average sense, the phenomena observed.

[4] "Etudes sur le mouvement des eaux," 2d Edition, 1863, pp. 58–68.

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flowing between regular banks. Does any fixed connection exist between the velocity of the mean of all the cubic feet passing a given section per second and the weight of sediment contained in that ideal cubic foot of water? If so, is the relation one of cause and effect? To the first of these questions Humphreys and Abbot give an emphatic negative,[1] based upon a comparison between the Carrollton and Columbus sediment and velocity curves. Captain Ead's criticism of this view was founded on a misconception.

He proves conclusively in his article that the total weight of sediment passing a given point in the river is proportional to the velocity of the current. This, however, was not the question at issue, and is settled by a moment's reflection. The debatable problem is: Does an increase of velocity increase the transporting power per cubic foot of water passing a given cross-section? The vital question in the problem of jetties is not as to the ability of the contracted stream to carry throughout their length and beyond the sediment contained per cubic foot in the water of the river above. Their success hinges upon the capacity of the stream to take an additional load per cubic foot from its increased velocity until the increasing depth has again established equilibrium. The consensus of opinion of writers seems to answer in the affirmative as opposed to the position of Humphreys and Abbot.

Partiot says[2] that sediment in floods follows the same law as the velocities, increasing up to the highest stage and decreasing afterwards.

Referring to the Missouri, Major Ruffner[3] says:

"The water is so heavily charged with sediment that decrease in velocity is immediately followed by a deposit, but the converse of scour following an increase of velocity, although apparent, is not so well marked nor so extensive. * * * When from any cause the velocity of the current is suddenly increased, the most rapid erosion takes place; and the greatest deposit occurs when the velocity is suddenly checked."

Captain Eads and Mr. Corthell[4] state that the current of the Mississippi cannot be checked in the slightest degree in flood time, when its waters are heavily charged, without causing a deposit; that an

[1] See p. 264.

[2] "Sables de la Loire," p. 22.

[3] "Improvement of Non-Tidal Rivers," 1886, pp. 78, 138.

[4] *Transactions* American Society of Civil Engineers, Vol. xi, pp. 262, 263.

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iron netting, with meshes 1 ft. square, set in a shoal on the Missouri, caused a deposit of 16 ft. in one flood. Mr. Ockerson[1] claims that the river is not always fully charged with sediment and so may at times be retarded to some extent without deposit.

Major Allan Cunningham[2] states that, in the Ganges canal, measurements have shown that silt density is independent of velocity.

Mr. R. E. McMath[3] says the cause of suspension commonly varies with the velocity.

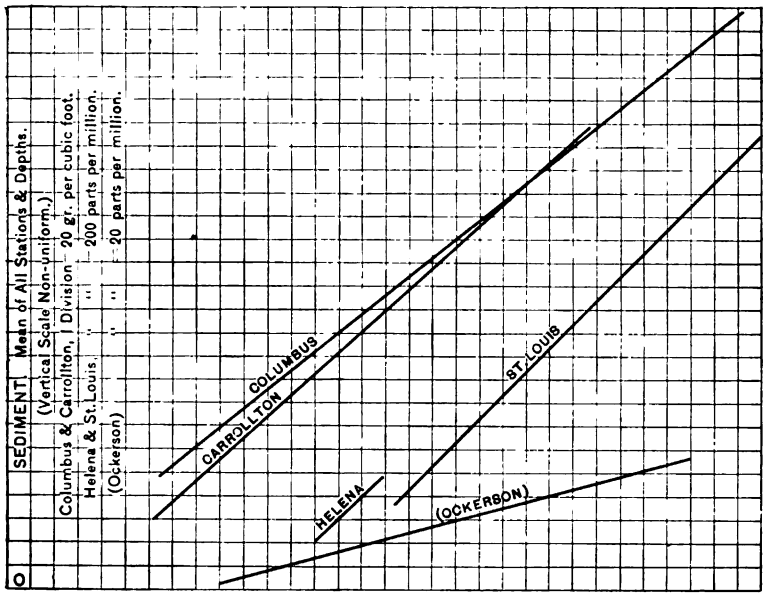

FIG. 2.

These views represent all phases of opinion. General statements are not convincing so it is advisable to analyze the available measurements.

Fig. 2 gives a graphic representation of five different series of measurements in the Mississippi. Other data at hand are unaccompanied by velocity determinations. These curves are plotted with mean velocities as abscissas and amounts of sediment as ordinates. As

[1] *Transactions*, American Society of Civil Engineers, Vol. xi, p. 273.

[2] "Roorkee Hydraulic Experiments." See *Proceedings* of the Institution of Civil Engineers; Vol. 71, p. 35.

[3] "Silt Movement by the Mississippi." *Van Nostrand's Engineering Magazine*, 1883, p. 38.

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in Fig. 1 they are not suitable for quantitative comparison. There is no co-ordination in vertical scales. Each represents only the general trend of direction of the points from which it is drawn. The points for the Columbus and Carrollton curves were the most scattering, as would be expected from an examination of the same values plotted in Plates XII and XIII of Humphreys and Abbot's "Report on the Mississippi." There, time is introduced so as to form two curves. Here, a general relation is expressed by one curve, omitting the time element and plotting each observation as a separate point fixed by its sediment and velocity co-ordinates.

An examination of the two curves as plotted by Humphreys and Abbot seems to indicate a relation, especially in the case of Carrollton where samples were taken from three depths and averaged. The relation is not, however, invariable. From the points as plotted here there can be no doubt of the existence of a law.

Another set of measurements was instituted at Carrollton by the Mississippi River Commission in 1879–80 which removes all doubt as to the existence of a relation between velocity and sediment at that point.[1]

The investigations made by this commission at Prescott, Winona, Clayton, Grafton, Hannibal and St. Louis in 1880 and 1881 are the most extensive ever conducted and offer the data for plotting additional curves.[2] Those made by the Missouri Commission at St. Charles[3] in 1879–1880 offer similar facilities.

Of the curves shown in Fig. 2, the most reliable are those representing the Helena and St. Louis measurements because of the improved methods used. In both these cases the law is clearly marked. The curve marked Ockerson is plotted from some measurements published by J. A. Ockerson,[4] M. Am. Soc. C. E.

In Fig. 2 it might seem reasonable to have started each curve from the origin on the ground that stagnant water would carry no sediment. On the other hand, such a proceeding would have been open to objection as an *argumentum in circulo*, and so was avoided. The curves, however, place themselves in a significant arrangement.

[1] See "Report of the Chief of Engineers, United States Army," 1883, III, pp. 2209–2266 and Plate VIII.

[2] These curves have all been plotted by the author and show the same result as those drawn on Fig. 2.

[3] "Report of the Chief of Engineers, United States Army," 1887, IV, pp. 3090–3096.

[4] *Transactions* of the American Society of Civil Engineers, Vol. xi, p. 273. Mr. Ockerson's values for sediment as given above show a marked departure from other determinations in the Mississippi. However, as only the relative quantities are required here, they are used without question.

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The curves are believed to show that the sediment by weight in a cubic foot of water does obey a general law of increase with the velocity in the Mississippi River. As such a relation has been largely conceded to exist in other rivers it is believed that it is general. That instant deposit always follow the least retardation is not proved or claimed.

4. Distribution of Sediment in the Cross-Section.

Distribution of Sediment in the Vertical.—Sufficient data is at hand to settle this question in its general bearing. That it has been a matter of dispute is shown by the different opinions expressed.

M. Baumgarten,[1] from measurements in the Garonne, came to the decision that surface measurements were a fair index of the amount of sediment at all depths.

Herr Blohm[2] quotes a number of varying opinions from English, German and Italian engineers; but, from his own measurements in the Elbe at Harburg, finds somewhat of an excess in the surface quantities. He, however, gives it as his opinion that the distribution of sediment is about equal throughout.

Andrew Brown[3] decided, after repeated trials, that sediment in the Mississippi was equally distributed at all depths, provided the samples were taken in the main current.

M. Partiot[4] takes the other view, stating that measurements in the Loire have shown the ratio of surface, middle and bottom quantities to the mean of all, to be represented by the numbers 90, 100 and 110.

M. Surell[5] found that the silt intensity in the Rhone increased rapidly with distance from the surface. He expressed the ratio between surface and bottom amounts by the relation of the numbers 100 and 188.

In the measurements on the Mississippi at Columbus,[6] only surface specimens were used, the ratio 100 to 120 being used to reduce the surface values to the mean for all depths as determined at Carrollton in

[1] "Navigation Fluviale, Garonne." M. Baumgarten. *Annales des Ponts et Chaussées*, 1842, 2, p. 49. See also page 244 of this paper.

[2] Ueber die in fliessenden Wasser suspendirt enthaltenen Sinkstoffe." Baurath Blohm. *Zeitschrift des Architekten und Ingenieur Vereins*, Hanover, 1867, p. 276.

[3] Humphreys and Abbot's "Report on the Mississippi," 1876, p. 143.

[4] "Sables de la Loire," 1871, p. 24.

[5] M. Guérard on "Mouth of the River Rhone." See *Proceedings* of the Institution of Civil Engineers, Vol. 82, p. 309.

[6] See Humphreys and Abbot's "Report on the Mississippi," 1876, p. 136.

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1851–1852, The following table has been prepared from all the data which could be collected upon this matter :

DISTRIBUTION OF SEDIMENT WITH REGARD TO DEPTH.

Place of observation and observer.	Date.	Reference.	PARTS OF SEDIMENT IN 1 000 000 PARTS OF WATER AT THE DEPTHS INDICATED.		
			Surface. (Mean.)	Mid depth. (Mean.)	Bottom (usually 1 ft. above). (Mean.)
Mississippi at St. Louis—McMath	1879	*Van Nostrand Eng. Mag.*, 1883	1 847	2 009	2 117
Mississippi at Helena—Johnson	1879	Rep't Chf. of Eng'rs U. S. A., 1879, III.	799	...	1 266
Mississippi at Carrollton—Forshey	1851-52	Rep't on Miss. Humphreys and Abbot, 1861	648	802	842
Sacramento at Kerscheval's—Le Conte	1879	Rep't Chf. of Eng'rs U. S. A, 1879, II.	5 525	9 051	5 618
Mississippi at Prescott—Miss. River Com.	1880-81	Rep't Chf. of Eng'rs U. S. A., 1883, III.	123	157	159
Mississippi at Winona—Miss. River Com.	1880-81	Rep't Chf. of Eng'rs U. S. A., 1883, III.	34	32	36
Mississippi at Clayton—Miss. River Com.	1880-81	Rep't Chf. of Eng'rs U. S. A, 1883, III.	40	42	41
Mississippi at Hannibal—Miss. River Com.	1880-81	Rep't Chf. of Eng'rs U. S. A., 1883, III.	165	206	224
Mississippi at Grafton—Miss. River Com.	1880-81	Rep't Chf. of Eng'rs U. S. A., 1883, III.	319	322	345
Mississippi at St. Louis—Miss. River Com.	1880-81	Rep't Chf. of Eng'rs U. S. A., 1883, III.	686	906	995
Mississippi at Carrollton—Miss. River Com.	1879-80	Rep't Chf of Eng'rs U. S. A., 1883, III.	"The means of surface, mid-depth and bottom observations are in the ratio of 100, 144 and 183, respectively." Vide Rep't Chf. Eng'r, 1883, III, p. 2216.		
Missouri at St. Charles—Missouri River Com.	1879	Rep't Chf of Eng'rs U. S. A., 1887, IV.	2 418	2 473	2 548
Garonne—Baumgarten	1847	Annales des Ponts et Chaussées, 1848, II.	The surface, mid-depth and bottom quantities are in the ratio of the numbers 100, 141 and 125.		
Elbe at Harburg—Blohm.	1837-54	Zeitschrift des Arch. und Ing. Ver., Hanover, 1867	Surface mid-depth and bottom quantities are in the ratio of the numbers 100, 93.8 and 98 2.		

A study of the table shows only one case in which the sediment per cubic unit is not greater at the bottom than at the surface. This is that of the Elbe at Harburg. There are two cases where the mid-depth amounts are less than the surface values, the Mississippi at Winona and the Elbe at Harburg. There are three cases where the bottom values are less than those at mid-depth, the Sacramento at Kerscheval's, the Mississippi at Clayton, and the Garonne.

In all other cases there is a marked increase from surface to bottom. When it is considered how extensive a range of measurements is represented in these means and due weight is given to the careful and far-

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reaching observations of the Mississippi and Missouri Commissions, it is thought that the law of increase from surface to bottom may be considered as established.

The table shows two other facts. *First*, that surface observations are not an accurate index of the mean amount of sediment. *Second*, that no general coefficient should be used to reduce surface observations to mean values for all depths, since this coefficient will vary in different rivers and for different stages of the same river.

It was hoped that data might be found from which a relation could be established between the form of the velocity and sediment curve in the same vertical. Measurements at all points of the vertical are needed with simultaneous velocity observations at the same depths. The only ones obtained at all partaking of this nature are those in the Sacramento River, which are too limited to be of service.[1] The observations at Carrollton in 1880 showed little change in the surface sediment with change of stage.

Law of Distribution in the Horizontal.—In this study the data available are still more limited. Of the extensive measurements carried on by the Mississippi and Missouri Commissions, the author can find no single case where the samples from the eight positions in the transverse sense were kept separate and the weights published. Mr. Seddon[2] states that nothing of special interest was shown by these measurements at St. Charles, so the data were not published. The Carrollton observations[3] of 1880 are said to have shown a uniform distribution from side to side of the channel and so are not published. Major Cunningham[4] obtained no data from which a law could be predicated in his measurements on the Ganges Canal.

Those at Columbus in 1858 are not printed in detail, but the Carrollton observations of 1851–52 give the single satisfactory set. The measurements made in 1879 by Mr. McMath at St. Louis are not published in detail, but give some facts of interest. The last two sets are represented in the table on the next page by their means for such positions as are indicated.

[1] An article by C. C. Babb, Jun. Am. Soc. C. E., abstracted in *Engineering News*, August 10th, 1893, is said to give curves representing sediment at different depths in the Potomac River.

[2] "Report of the Chief of Engineers, United States Army," 1887, IV, p. 3090.

[3] "Report of the Chief of Engineers, United States Army," 1883, III, p. 2216.

[4] *Proceedings* of the Institution of Civil Engineers, Vol. 71, p. 34, "Roorkee Hydraulic Experiments."

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DISTRIBUTION OF SEDIMENT IN THE HORIZONTAL.

Place of observation and observer.	Date.	Reference.	PARTS OF SEDIMENT IN 1 000 000 PARTS OF WATER AT THE POSITIONS INDICATED.		
			Bank.	Middle.	Bank.
Mississippi at Carrollton —Forshey	1851-52	Rep't on Miss., Humphreys and Abbot, 1861	542 (300 ft. from e. bank).....	573	543 (400 ft. from w. bank).
Mississippi at St. Louis - McMath	1879	*Van Nostrand Eng. Mag.*, 1883.	Mean of sediment in the river, 2 062 parts; mean of sediment, 281 ft. from Mo. side, 2 736 parts; mean of sediment 1 291 ft. from Ill. side, 1 523 parts.		

These meager measurements are offered rather to show the need of attention to this matter than as a proof that the maximum of sediment is near the thread of the stream. Mr. McMath[1] found the maximum in nearly every case several hundred feet on the Missouri side of the line of maximum velocity. This could, perhaps, be accounted for by the fact of the Missouri water being more highly charged since its sediment is finer. If the Ohio and Mississippi flow side by side without mingling their waters[2] it may be that a similar phenomenon occurs at St. Louis.

Surface Convexity and the Lateral Movement of Suspended Matter.— It has been stated[3] that crevasses in the Mississippi show a marked swelling or convexity at the thread of the current, where it crosses the levee, and that this convexity, which is due to the excess of velocity, has the effect of drawing floats to a narrow line at the filament of maximum velocity.

Major Cunningham[4] could measure no sensible curvature at the center of the Ganges Canal. He had expected to find such a swelling on the ground that increased velocity would be accompanied by decreased pressure. He quotes the statement of General Rundall that the surface of the Godavery and Mahanuddy Rivers was obviously con-

[1] "Silt Movement by the Mississippi," R. E. McMath. *Van Nostrand's Engineering Magazine*, Vol. 28, 1883, p. 36.

[2] See "Report on the Mississippi," Humphreys and Abbot, 1876, p. 136. Compare, also, the statement made by McMath in "The Mississippi as a Silt-Bearer," *Van Nostrand's Engineering Magazine*, 1879, p. 222. The Missouri water was found to contain 2½ times the amount of sediment carried by the water of the upper Mississippi where the two streams were running side by side, with a common velocity, past Bissell's Point.

[3] "Report on the Mississippi," Humphreys and Abbot, 1876, p. 284.

[4] *Proceedings* of the Institution of Civil Engineers, Vol. 71, pp. 11 and 12.

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cave, plane or convex, according as the rivers were falling, stationary or rising.

Mr. Flamant[1] shows that there should be no such difference in pressure since a fluid transmits pressure equally in all directions and the surface should be level for permanent motion. He explains the convexity measured by Baumgarten[2] in the Garonne in one case on the ground that it did not correspond to a permanent state, but to a condition of rise when the center would show the increase before the sides.

M. Baumgarten[3] concludes from his two measurements on the Garonne that the changes are scarcely sensible.

The experiments of Darcy and Bazin[4] showed no results which could be said to express a law.

Prof. De Volson Wood[5] states that the water is highest where the velocity is greatest, but gives no substantiation for the statement.

M. Debauve[6] accepts the idea as true and explains its apparent disagreement with the hydrostatic law on the ground of viscosity, the water tending to the form which will offer the least resistance at the banks.

Major Cunningham[7] found that surface floats placed near the banks were uniformly drawn out to the thread of the stream, while subsurface floats maintained a direction sensibly parallel to the banks.

Mr. McMath[8] holds that suspended material is borne from the sides toward the center. The same statement is made by Dupuit[9] with reference to bodies floating at the surface.

M. Lagrene[10] considers the upper surface in straight sections to be sensibly horizontal so far as present knowledge reaches.

In general it may be said that the movement of surface floats shows a tendency toward the line of maximum velocity. That the difference

[1] See *Annales des Ponts et Chaussées*, 1882, IV, p. 56. Also *Proceedings* of the Institution of Civil Engineers, Vol. 71, p. 66.

[2] See *Proceedings* of the Institution of Civil Engineers, Vol. 71, p. 12, or *Annales des Ponts et Chaussées*, 1848, 2, pp. 28–30.

[3] *Annales des Ponts et Chaussées*, 1848, 2, p. 30.

[4] See "Recherches Experimentales." Darcy and Bazin. Plates XIX to XXVI. Also *Proceedings* of the Institution of Civil Engineers, Vol. 71, p. 12.

[5] *Van Nostrand's Engineering Magazine*, 1879, p. 370. He claims that the cause lies in the reduction of pressure with increase of velocity.

[6] "Navigation Fluviale et Maritime." V. Debauve.

[7] *Proceedings* of the Institution of Civil Engineers, Vol. 71, p. 23. "Roorkee Hydraulic Experiments."

[8] *Van Nostrand's Engineering Magazine*, Vol. 28, p. 35.

[9] "Etudes sur le Mouvement des Eaux," p. 218.

[10] "Cours de Navigation Intérieure," p. 53.

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in height of surface between center and sides is a concomitant phenomenon cannot be said to be proved. There are many statements that the fact exists, but such measurements as are available fail to substantiate them.

Does Suspended Matter Move Faster Than the Current?—The question of the existence of a relative velocity between surface floats and the surface current may be said to be settled. It has been considered a source of error in the measurement of surface velocities by floats for some time past. The velocity of the float is greater than the mean velocity of the displaced water. For the same float, this relative velocity will increase with increase in depth of flotation.[1]

Major Cunningham[2] takes the contrary view, claiming that the velocity of a submerged rod is somewhat less than the mean velocity past its immersed length, so that it should only extend $\frac{94}{100}$ of the total depth to the bottom in order to give a true indication of the mean velocity in a vertical.

The experiments of M. Du Boys[3] upon the Rhone may be said to settle the matter beyond dispute. He found from experiments upon boats that they moved sensibly faster than the current, and that this relative velocity varied with the form of the boat and increased with its size and with the velocity of the current.

Experiments by M. Bérard,[4] in 1886, on an artificial canal, showed that a float moved faster than the surface velocity, but almost identically at the same rate as the mean velocity of the displaced water.

M. Du Boys,[5] however, gives the case of a boat which had a velocity of 4.46 m. in a current whose surface velocity was only 2.75 m. These differences are too great to be attributed to the fact that the maximum velocity is below the surface.

A block of wood and a floating canal-boat will not remain side by side in a river, but gradually draw away from each other, the boat taking the lead.

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[1] Noted and incorrectly explained in *Zeitschrift des Architekten- und Ingenieur- Vereins*, Hanover, 1887, p. 628.

[2] "Roorkee Hydraulic Experiments." *Proceedings* of the Institution of Civil Engineers, Vol. 71, p. 22.

[3] "La Marche des Bateaux dans les Courants Rapides." *Annales des Ponts et Chaussées*, 1886, I, pp. 199–242.

[4] "Marche des Flotteurs dans les Courants," *Annales des Ponts et Chaussées*, 1886, II, p. 30.

[5] "Hydraulique." Flamant, p. 299.

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The Question of Lateral and Vertical Flow in Rivers.—Islands are said to be formed at the center of rivers at the expense of the banks, indicating a transverse flow of the particles toward the middle. Is there a regular lateral flow of the water obeying a law as fixed as that which determines the general law of translation? If so, it will enter as an important factor in governing sedimentary movements. Prof. James Thomson[1] has demonstrated the presence of such a flow at curves where the motion at the surface is outward and at the bed is inward. This offers a suitable explanation of the cause of shoals opposite sharp bends. In a regular channel centrifugal force does not enter in to produce these effects. Are they present? Mr. McMath says[2] that observation has failed to detect any division of a stream into ascending or descending areas other than local motions due to eddies. Major Cunningham[3] claims to have detected a surface flow toward the center in the Ganges Canal as indicated by the motion of the floats near the banks, while a sub-surface flow in the contrary sense was indirectly shown by the fact that the deeply immersed floats showed no general transverse tendency.

Experiments made by J. B. Francis[4] in regular canals with whitewash discharged through a tube opening near the bed, seemed to show a vertical movement of the water from the bed to the surface, the whitewash appearing at a distance down-stream varying from 10 to 30 times the depth. Mr. Francis offers this vertical movement as the cause of suspension of sediment.

Prof. De Volson Wood,[5] upon the ground of these observations of Mr. Francis, adopts the idea of a lateral surface movement toward the banks, with a corresponding bottom current toward the center.

F. P. Stearns, M. Am. Soc. C. E.,[6] on the other hand, holds the view that there is a motion of bottom water to the surface at the sides and toward the center at the surface.

The observations of Mr. Francis might seem to be corroborated by the positive statement[7] that the Mississippi water is constantly rising from the bed to the surface.

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[1] "Encyclopedia Britannica." Article Hydrodynamics, p. 498.
[2] "Silt Movement by the Mississippi," *Van Nostrand's Engineering Magazine*, Vol. XXVIII, p. 35.
[3] *Proceedings* of the Institution of Civil Engineers, Vol. LXXXII, pp. 23, 24. "Roorkee Hydraulic Experiments."
[4] *Transactions* of the American Society of Civil Engineers, Vol. VII, p. 109.
[5] *Van Nostrand's Engineering Magazine*, Vol. XXI, 1879, p. 369.
Also *Transactions* of the American Society of Civil Engineers, June 17th, 1879.
[6] *Transactions* of the American Society of Civil Engineers, Vol. XII, p. 331.
[7] "Report of the Chief of Engineers, United States Army," 1883, III, p. 2218.

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5. Solid Discharge of Rivers.

In many articles on river correction and in most of the treatises on river hydraulics will be found statements of the amount of silt carried by streams in various parts of the earth. The amounts are unreliable in some cases and in many are expressive of only suspended matter to the exclusion of the more or less extensive movement along the bottom. A collection of such non-homogeneous data is not deemed of value for the purposes of the present paper.[1]

Proportionate Amounts Suspended and Dragged.—The quantitative relation existing between the amounts moved at or near the bottom and those carried in free suspension has offered another opportunity for diversity of opinion.

Dupuit[2] looks upon transportation in suspension as the most important element, on the ground of the slight velocity of the materials dragged. Prof. Forshey[3] concludes that the matter pushed along the bed of the Mississippi forms about three-fourths of its total solid discharge.

M. Guérard[4] is persuaded that the greater portion of the solid matter discharged by the Rhone is pushed along its bed. The Mississippi River Commission,[5] after a series of careful measurements of sand waves at Carrollton, decided that not more than 0.8 of 1 per cent. of the total solid discharge was moved along the bed of the river at this point. Major Ruffner[6] states that the movement of material at the lower laminæ of the Missouri at St. Charles was believed to be as great as that in all the rest of the river. He refers[7] also to observa-

[1] The following list gives a few sources of information upon this point:
"Report on the Mississippi," Humphreys and Abbot, 1876, p. 146.
Zeitschrift des Arch.- und Ing - Vereins, Hanover, 1867, pp. 245–50.
Proceedings of the Institution of Civil Engineers, Vol. XXI, pp. 15, 27, 459; Vol. LIII, p. 18; Vol. LI, pp. 216, 217; Vol, LVII, pp. 272–4.
The Engineer, October 25th, 1889, p. 343.
Annales des Ponts et Chaussées, 1848, II, pp. 46–48; 1860, I, p. 137; 1860, II, p. 374; 1869, I, p. 588; 1871, I, p. 15.
"Canal and River Engineering," Stevenson, p. 318.
"Improvement of Non-Tidal Rivers," Ruffner.
"Cours d'Hydraulique Agricole et Urbaine." M. Bechmann, 1895, pp. 109, 120. Also treatises on general geology.

[2] "Etudes sur les Mouvements des Eaux," p. 216.

[3] *Proceedings* of the American Association for the Advancement of Science, Nashville, 1877. See also *Van Nostrand's Engineering Magazine*, Vol. XX, p. 227.

[4] *Proceedings* of the Institution of Civil Engineers, Vol. 82, p. 309.

[5] "Report of the Chief of Engineers, United States Army," 1883, III, p. 2218.

[6] "Improvement of Non-Tidal Rivers," p. 78.

[7] The same, p. 138.

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tions at Lake Providence, La., which indicated that a large amount was moving along the bed of the Mississippi at that point of which the moving sand waves represented only a small portion.

Captain Eads and Mr. Corthell[1] join with the Mississippi River Commission in the statement that nearly all the solid matter in the Mississippi is carried in suspension, while but a small proportion is dragged on the bottom.

It is believed that this last statement represents the actual case in most sedimentary rivers.

6. Tabulation of Observed Data on Dragging.

A tabulation of the best known results of experiment on velocities at at which dragging begins is given on the next page. Those given by Bouniceau appear to be taken from the results of Dubuat's and Telford's experiments. The others are believed to represent original measurements or computations. In some cases the published results do not state whether the velocity measured was that at surface, mid-depth or bottom.

Further statements of velocities are given by Weisbach, Unwin, Church, Bechmann and others, but they are all based upon the measurements detailed in this table. A limited collection of similar data is given in the Report of the Chief of Engineers, U. S. Army, 1885, I, pages 569–570.

The detail of Dubuat's and Sainjon's measurements has been given in a preceding part of this paper. The measurements in the Upper Rhine were made near Alt-Breisach, with a smooth river-bed. In the cases so marked, motion did not take place until the particles were subjected to a slight disturbance from the outside.

Login's experiments were made with a stream averaging $\frac{1}{4}$ in. in depth. Telford's velocities are those at which erosion begins. Blackwell's measurements have been referred to on page 248.

The materials are given as described by the experimenter. A careful set of measurements of these velocities, made with improved apparatus, is much needed.

Displacement of Crests of Sand Waves.—For purposes of comparison, a collection has been made on page 310 of measurements on sand waves.

[1] *Transactions* of the American Society of Civil Engineers, Vol. XI, p. 262.

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TABLE GIVING VELOCITIES OF CURRENT AT WHICH DRAGGING BEGINS.

Authority	Dubuat.	Telford.	Blackwell.	Sainjon.	Login.	Rhine measurements.	Zschokke.	Verein Hütte.	Bouniceau.
Reference	Hydraulique. Paris, 1786. p. 94.	Partiot in Annales des Ponts et Ch. 1871, I, p.34.	Proc. Inst. Civil Engineers, Vol.82, pp.47–50.	Partiot in Annales des Ponts et Ch. 1871, I, p.33.	Stevenson's Canal and River Engineering, p. 315.	Zeitschrift des A. und I. Vereins zu Hannover, 1884, p. 176.	Lectures, Zürich, 1895.	Ingenieurs Taschenbuch.	"Etudes sur la Navigation," 1845, p. 19.
REMARKS.	Bottom velocity. Feet per sec.	Bottom (?) velocity. Feet per sec.	Feet per second.	Bottom velocity. Feet per sec.	Bottom velocity. Feet per sec.	Feet per second.	Feet persecond	Bottom velocity. Feet per sec.	Bottom velocity. Ft. per sec.
Soft earth	.....	0.25	.....	.....	.....	.....	.....	.....	.....
Brick clay {allowed to settle ½ hour in water}	.....	.....	.....	.....	0.25	.....	.....	.....	0.26
Potter's clay	0.27	.....	.....	.....	.....	.....	.....	0.26	.....
Soft clay	.....	0.50	.....	.....	.....	.....	.....	0.52	0.49
Fresh-water sand	.....	.....	.....	.....	0.667	.....	.....	.....	.....
Large sand	0.71	.....	.....	.....	.....	.....	.....	.....	0.72
Vegetable soil	.....	.....	.....	.....	0.833	.....	.....	.....	.....
Sand	.....	1.00	.....	.....	.....	.....	.....	.....	0.98
Firm sand	.....	.....	.....	.....	.....	.....	.....	1.02	.....
Sea sand	.....	.....	.....	.....	1.103	.....	.....	.....	.....
Gravel (size of anise seed)	0.36	.....	.....	.....	.....	.....	.....	.....	0.36
Gravel (size of peas)	0.62	.....	.....	.....	2.000	2.46 (if disturbed)	0.62	.....	.....
Gravel (size of beans)	.....	.....	.....	.....	.....	2.95 (if disturbed)	1.066	.....	.....
Gravel (diameter, .008 ft.)	.....	.....	.....	0.82	.....	.....	.....	.....	.....
Gravel (diameter, .03 ft.)	.....	.....	.....	1.64	.....	.....	.....	.....	.....
Brickbat(contents,2.59 cu.ins.)	.....	.....	1.75–2.00	.....	.....	.....	.....	.....	.....
Gravel	.....	2.00	.....	.....	.....	.....	.....	.....	.....
Oolites (2.39 cu. ins.)	.....	.....	2.00–2.25	.....	.....	.....	.....	.....	.....
Slate (2.38 cu. ins.)	.....	.....	2.00–2.25	.....	.....	.....	.....	.....	.....
Sea pebbles (1.06 ins.diameter)	2.13	.....	.....	.....	.....	.....	.....	2.30	2.13
Brickbats (4.76 cu. ins)	.....	.....	2.25–2.50	.....	.....	.....	.....	.....	.....
Flints (1.96 cu. ins.)	.....	.....	2.50–2.75	.....	.....	.....	.....	.....	.....
Brickbats (18.5 cu. ins)	.....	.....	2.75–3.00	.....	.....	.....	.....	.....	.....
Oolites (17.68 cu. ins)	.....	.....	2.75–3.00	.....	.....	.....	.....	.....	.....
Slate (9.06 cu. ins.)	.....	.....	2.75–3.00	.....	.....	.....	.....	.....	.....

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Flints (10.37 cu. ins.)	.....	.....	3.00–3.26	.....	.....	.....	.....	.....
Pebbles	.....	3.00	.....	.....	.....	.....	.....	3.26
Boulders	.....	.....	.....	.....	.....	.....	.....	.....
Gravel (.085 ft. diameter)	.....	.....	.....	.....	.....	.....	.....	.....
Gravel (.13 ft. diameter)	.....	.....	.....	3.26	.....	3.09	3.08	.....
Gravel (size, hazel to walnut)	.....	.....	.....	.....	3.48 (if disturbed)	.....	.....	.....
Gravel (size, pigeon's egg)	.....	.....	.....	.....	3.67 (if disturbed)	.....	.....	.....
Smallest gravel	.....	.....	.....	.....	3.87	.....	.....	.....
Gravel (walnut size)	.....	.....	.....	.....	4.22	.....	.....	.....
Gravel (.17 ft. diameter)	.....	.....	.....	.....	.....	5.15	.....	.....
Gravel (weight, 2.0 grams)	.....	.....	.....	.....	4.92 (if disturbed)	.....	.....	3.94
Broken stone	.....	4.00	.....	.....	.....	.....	.....	.....
Gravel (weight, 2 500 grams)	.....	.....	.....	.....	5.90 (if disturbed)	.....	4.90	5.00
Conglomerate (soft schist)	.....	5.00	.....	.....	.....	.....	.....	.....
Gravel (.33 ft. diameter)	.....	.....	.....	4.92	.....	.....	.....	.....
Laminated rock	.....	6.00	.....	.....	.....	.....	6.00	6.00
Gravel (.56 ft. diameter)	.....	.....	.....	6.56	.....	.....	.....	.....
All gravel	.....	.....	.....	.....	6.56(if disturbed)	.....	.....	.....
Gravel (.13 cu. ft.)	.....	.....	.....	.....	.....	7.23	.....	.....
Gravel (1.25 ft. diameter)	.....	.....	.....	9.84	.....	.....	.....	.....
Hard rock	.....	10.00	.....	.....	.....	.....	10.36	9.84
Gravel (1.09 cu. ft.)	.....	.....	.....	.....	.....	10.29	.....	.....
Gravel (2.20 ft. diameter)	.....	.....	.....	13.12	.....	.....	.....	.....
Gravel (2.18 cu. ft.)	.....	.....	.....	.....	.....	15.44	.....	.....

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Tabulated Data on Movement of Sand Waves.

Locality.	Date.	Observer.	Velocity of Stream. Feet per Second.			Average depth of stream. Feet.	Material.	Dimensions of Wave. Feet.		Velocity of crest. Feet per day.	Reference.
			Surface.	Mean.	Bottom.			Height.	Length.		
Mississippi, at Helena.....	March to June, 1879.	J. B. Johnson	.....	3.58	.....	13 to 30	Sand.	5.0	326.0	17.75	Rep't Chf. of Engrs., U. S. A., 1879, p. 1967.
Loire, at Angers	June to Nov., 1858.	.....	.....	.....	.....	Scale read from — 0.3 to + 0.85.	Sand.	.....	.....	7.35	Debauve Nav. Fluviale, p. 135.
Loire, at Angers	Nov., 1858, to June, 1859.	.....	.....	.....	.....	Scale read from + 0.80 to + 3.75.	Sand.	.....	.....	29.52	
Mississippi, at Lake Providence	Nov., 1879, to Nov., 1880.	Hider.....	.....	.....	.....	(High water) 76.	Sand.	13.0	470	22.5	Rept. of Chf. of Engrs., U. S. A., 1883, pp. 2197-98, Appendix S. S.
"	"	"	.....	.....	.....	(25 ft. below high water) 46.	"	6.6	252	9.7	
"	"	"	.....	.....	.....	(High water) 80 to 100.	"	6.9	332	42.	
"	"	"	.....	.....	.....	(25 ft. below high water).	"	7.6	325	10.8	
Sacramento, at Kerscheval's..	March, 1879....	Le Conte.....	5.50	.....	(4.48 (2 to 7 ft. above bottom).	20 to 25 ft.	Sand.	2	13	.....	Report of Chf. of Engrs., 1879, p. 1759.
Garonne, at Colde-Fer	1840-43.....	Baumgarten..	.....	6.5 to 8.0	.....	.....	Gravel.	4.2	.....	(82 (ft. per year).	Annales des Ponts et Chaussées, 1848, 2.
Loire	1858-59.....	Partiot.....	.....	.....	.....	.....	Sand.	.....	.....	36.0	Sables de la Loire. Partiot. Annales I, 1871.
"	"	"	.....	.....	.....	.....	"	.....	.....	16.4	
"	"	"	.....	.....	.....	.....	"	.....	.....	16.4	
"	"	"	.....	.....	.....	.....	"	.....	.....	6.56	
Loire, at Nantes	1858.....	Duval.....	.....	.....	.....	.....	Sand.	.....	.....	4 2	Partiot. Annales des Ponts et Chaussées, 1871, I, p. 43.
"	"	"	.....	.....	.....	.....	"	.....	.....	11.2	

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Additional measurements, made by M. Sainjon, are given at page 257 of this paper.

In general, it may be said that the displacement of these wave crests increases with the stage and with the velocity, at least up to a certain point of bottom velocity. Their form and motion is most regular at times when the river is neither rising nor falling. Their motion is most rapid when the river is rising. They are largest in deep water.

7. Velocities of Vertical Current Required to Keep Particles Suspended.

A particle whose specific gravity is greater than 1 will sink in quiescent water when viscosity is overcome. It will, when falling freely from rest, after a very short period of acceleration, attain a state of velocity practically uniform. The same particle should be kept suspended indefinitely by a constant current, whose vertical component has a velocity equal to that asymptotically approached by the falling particle.

Experiments have been made to determine the velocities of currents necessary to keep particles of various sizes and specific gravities suspended when acting in a direction opposed to gravity.

The table[1] on page 312 gives the results of experiments made by M. Thoulet. It represents the maximum velocities in millimeters per second of, and the thrusts in milligrams exercised by, currents capable of holding suspended, at a fixed point in a tube, spherical grains of radius varying from 0.1 to 2.5 mm., and of densities, in air, between 1.5 and 4.0. The velocities are taken from diagrams constructed from results of actual experiment. The thrusts P are computed on the supposition that a current with a velocity v, in millimeters, which holds in suspension a body of density γ', exercises against the body a thrust

$$P = \frac{4}{3}\pi r^3 (\gamma' - 1)$$ (in milligrams, where r is expressed in millimeters).

The range of values for (r) and (γ') is sufficient to cover all ordinary mineral grains.

There can be little doubt that the velocity of vertical current or the amount of vertical thrust required for the suspension of ordinary grains is very slight. Col. Mansfield reports[2] observations on grains of sand allowed to fall from rest in water. The maximum velocity

[1] For further details see p. 269; also *Annales des Mines*, 1884, I, p. 521.

[2] "Report of Chief of Engineers, United States Army," 1886, pp. 1298–1299.

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Radius (r) of grains in millimeters.	Velocity (v) of the current in millimeters per second. Thrust (P) in milligrams exercised against the spherical grain of density (γ').											
	$\gamma' = 1.5$ v	P	$\gamma' = 2.0$ v	P	$\gamma' = 2.5$ v	P	$\gamma' = 3.0$ v	P	$\gamma' = 3.5$ v	P	$\gamma' = 4.0$ v	P
0.1.....	13	0.0002	23	0.004	32	0.006	40	0.008	47	0.01	53	0.01
0.2.....	23	0.002	40	0.03	55	0.05	69	0.07	79	0.08	85	0.10
0.3.....	32	0.006	57	0.11	78	0.17	94	0.23	106	0.28	116	0.34
0.4.....	40	0.01	70	0.27	94	0.40	114	0.54	129	0.67	143	0.80
0.5.....	51	0.03	82	0.52	107	0.78	132	1.05	151	1.31	168	1.56
0.6.....	57	0.04	92	0.90	120	1.36	147	1.81	168	2.26	187	2.72
0.7.....	61	0.07	100	1.44	133	2.15	163	2.87	185	3.59	205	4.30
0.8.....	65	0.11	108	2.14	142	3.22	174	4.29	198	5.36	221	6.44
0.9.....	68	0.15	116	3.05	152	4.58	185	6.11	211	7.63	237	9.16
1.0.....	72	0.21	123	4.19	161	6.28	195	8.38	224	10.47	252	12.56
1.1.....	75	0.28	128	5.57	169	8.36	203	11.15	234	13.94	264	16.72
1.2.....	78	0.36	133	7.24	175	10 86	211	14.48	243	18.10	273	21.72
1.3.....	80	0.46	137	9.20	180	13.80	218	18.40	252	23.01	281	27.60
1.4.....	82	0.57	141	11.49	184	17.24	224	22.99	260	28.73	288	34.48
1.5.....	84	0.71	144	14.13	188	21.21	229	28.27	267	35.34	295	42.42
1.6.....	86	0.86	147	17.15	192	25.74	234	34.31	271	42.89	300	51.48
1.7.....	88	1.03	149	20.57	195	30.87	237	41.16	274	51.45	304	61.74
1.8.....	90	1.22	151	24.42	197	36.64	240	48.86	277	61.07	308	73.28
1.9.....	92	1.44	153	28.72	199	43.10	243	57.46	280	71.83	312	86.20
2.0.....	94	1.67	154	33.50	201	50.27	246	67.02	283	83.78	315	100.54
2.1.....	95	1.94	155	38.78	202	58.19	249	77.58	286	96.98	317	116.38
2.2.....	96	2.23	156	44.59	203	66.90	251	89.20	286	111.50	319	133.80
2.3.....	97	2.55	156	50.95	204	76.45	253	101.93	290	127.41	321	152.90
2.4.....	98	2.89	156	57.89	205	86.86	254	115.81	291	144.76	323	173.72
2.5.....	98	3.27	156	65.44	206	98.17	255	130.90	292	163.62	324	196.35

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reached at the bottom showed that a vertical current of 1 or 2 ins. per second would keep them suspended.

Mr. T. E. Login,[1] found that the following materials had a rate of sinking in water of, and consequently would be sustained by a vertical current of, a velocity equal to the numbers given below:

Materials.	Velocity required. (Feet per second.)
Brick clay (mixed with water and allowed half hour to settle)	0.009
Fresh water sand	0.166
Sea sand	0.196
Rounded pebbles (size of peas)	1.000

Diagrams have been prepared by Richards[2] and Woodward to show the relations existing between the specific gravity, diameter and velocity of fall in water of various minerals. They are constructed from a formula given by Rittinger.[3]

$$V = 2.44 \sqrt{D(\delta - 1)}$$

where V = velocity of fall in meters per second.

D = diameter of particles in meters.

δ = specific gravity of the mineral.

These diagrams are extended to cover specific gravities between 1 and 15, diameters between 0 and 0.06 m., and velocities of fall up to 2.3 m. per second.

8. Results Expressed in the Form of a Series of Propositions.

The following propositions are believed to express the main facts, so far as they are known at present, which must be recognized by any broad theory of the cause of the suspension of sediment:

1. The movement of solids by water currents may take place by dragging, by intermittent suspension or by continuous suspension.

2. Motion in each of the three ways is increased with increase of depth; yet the depth itself can only affect the intermittent motion.

3. Motion by each of the three ways is increased by increase in mean velocity.

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[1] *Proceedings* of the Royal Society of Edinburgh, Vol. III, p. 475.
Also "Canal and River Engineering," Stevenson, p. 315.

[2] "The Velocity of Bodies of Different Specific Gravity Falling in Water." Richards and Woodward. *Transactions* of the American Institute of Mining Engineers, 1890, Vol. 18, pp. 644-648.

[3] Rittinger's "Aufbereitungskunde," 1867, p. 195..

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4. The presence of sediment in a stream decreases its mean velocity.

5. Dragging as well as suspending power increases with the heaviness of the liquid and with its coefficient of viscosity.

6. By far the greater amount of solid material transported by sedimentary rivers is carried in continuous suspension.

7. Sand waves of considerable size are formed in the larger rivers. Their motion down stream is slow, increasing in rate up to a certain critical value of bottom velocity and then decreasing with further increase.

8. The fineness of material in suspension increases from the rise to the embouchure of rivers.

9. The weight of suspended material per cubic unit of water decreases from rise to embouchure.

10. Increase in vortex motion increases power of transport.

11. Continuous vertical or lateral currents which could not be explained by local causes in rivers have not yet been proved.

12. The phenomenon of suspension requires for its explanation a continuous upward force. Intermittent forces are not sufficient, although the intensity of the force may vary.

13. Bodies suspended in flowing water, either intermittently or continuously, tend to acquire a velocity greater than that of the water surrounding them.

14. The transportation of material consumes part of the energy of any silt-bearing stream. A greater load of fine particles than of coarse can be carried with the same expenditure of energy.

Part III.—Discussion of Theories of Suspension.

The explanations offered as to the cause of this phenomenon, as detailed in the preceding pages, fall naturally into four categories. It will be well to examine each in the light of the facts at hand before drawing conclusions as to their relative value.

(*a*) *Theory of a Continuous Upward Flow.*—J. B. Francis[1] Past-President Am. Soc. C. E., sought to explain the suspension of sediment in streams by the presence of a continuous upward flow of the water at the bed. This flow was believed to be proved by experiments made by liberating whitewash at the bed of two different channels and noting its appearance at the surface at distances down-

[1] *Transactions* of the American Society of Civil Engineers, May and June, 1878.

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stream varying from 10 to 30 times the depth. The experiments of M. Thoulet[1] have shown how slight an upward current is required to produce this effect. Were such a resultant upward current proved, it might offer a satisfactory explanation of the phenomenon. There are, however, certain objections to this view which seem to render it inadmissible. Considering a long stretch of river, a steady flow from bed to surface at the center must be accompanied by a corresponding flow outward to the sides and a downward reverse flow along the bed from the sides to the center. Such a flow has not been observed. On the contrary several writers[2] have advanced the idea of a flow in the contrary sense, though this, too, must be regarded as yet unproved.

Again, even were the flow from bed to surface at center and outward to the sides proved, it would constitute no explanation of continuous suspension, unless the velocities of flow at center and sides were unequal, since it would give no resultant upward thrust. The algebraic sum of the vertical components of all the internal movements of the liquid should reduce to zero in the stretch considered.

Proofs of flow from bed to surface or from sides to center based upon indications offered by floating substances—the case in both Mr. Francis' and Major Cunningham's experiments—can not be considered conclusive. In the one case the movement might be attributed to local currents induced by the presence of the experimental apparatus. In both cases, the fact that a floating body tends to move faster than the surrounding medium and would follow the line of least resistance would cause floating substances to indicate currents which did not actually exist. This matter will be referred to later.

(*b*) *Discontinuous Upward Flow or Eddy Theory.*—There are certain writers who attribute the entire suspending power of streams to vertical currents incident to the complex eddying motion induced by the asperities of the bed. The numerous valuable articles[3] by R. E. McMath, M. Am. Soc. C. E., have taken the strongest position on

[1] See p. 270.

[2] See *Proceedings* of the Institution of Civil Engineers, Vol. 71, p. 23 and discussion. Also *Transactions* American Society of Civil Engineers, Vol. 12, p. 333.

[3] "Silt Movement by the Mississippi." *Van Nostrand's Engineering Magazine*, Vol. 28, 1883, p. 32.

"The Mississippi as a Silt Bearer." *Van Nostrand's Engineering Magazine*, Vol. 20, 1879, p. 218.

"Theory and application of the Permeable System of Works for the Improvement of Silt-Bearing Rivers." *Engineering News*, November 1st, 1879.

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this line. Upon the basis that the fact of suspension itself is an evidence of a resultant upward thrust, Mr. McMath states his position in these words:[1]

"This third hypothesis goes but one step farther in ascribing to the irregular movements the whole work of suspension upon the ground that a cause known to exist wherever the fact to be accounted for occurs, and admitted to be efficient, must be considered the sole cause unless a co-working agency is known, or the cause is insufficient to produce the observed result. Observation readily detects whirls, boils and eddies in the act of bringing water and suspended material from the bottom to the surface and laterally from the sides to the center of the river. Observation has never detected any other cause incident to the flow of streams which produced these effects."[2]

The efficiency of vortices in producing local results has passed beyond the range of controversy. M. Boussinesq[3] divides them into two classes; first, those in which the constituent water is constantly changing as the result of an axial flow which increases in velocity as the radius of curvature decreases; second, those in which the constituent water remains the same, being formed during an interval of time by a definite mass of fluid and manifesting themselves in a complex rotation.

A case of the first kind is the common vortex over the discharge pipe of a hand basin. One of the second is seen in the experiment of the revolving glass or basin of water described earlier in this paper in the experiments of M. Fargue and M. Gallois.

In both forms of vortex the tangential velocity increases with approach to the axis in contradistinction to the case of a rotating solid. In each case solid material obeys an impulse toward the axis whether there be a flow of liquid in that direction or not. This is due to the tendency of the solid to move faster than the liquid and in so doing to follow the line of least resistance, *i. e.*, the line where differences of velocity are least. In the first form an additional impulse toward the axis is received from the current setting in that direction. These phenomena are clearly shown in the experiments referred to, though

[1] "Silt Movement by the Mississippi." *Van Nostrand's Engineering Magazine*, Vol. 28, p. 35.

[2] "The suspension of matter involves an upward motion of the water or medium in which such matter is suspended, if in no other way, by the law of adhesion. * * * Upward motion of the suspending medium involves of necessity a downward movement of equal volume, but not necessarily of equal velocity." McMath in *Engineering News*, November 1st, 1879.

[3] "Essai sur la Théorie des Eaux Courantes," p. 616.

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M. Gallois'[1] explanation is at variance with that given above. He takes the position that the particles are impelled toward the center because the relative velocities are greatest at that point. This is incompatible with the theory that the standard form of a vertical velocity curve in streams is a parabola. If the friction initiated at the sides exercises its retarding effect progressively toward the center, as is believed to be the case in rivers, the absolute velocity near the axis of the vortex should be greater, but the relative velocities of the fluid filaments should be a maximum near the sides. In other words the solids would move toward the axis of the vortex with radial velocities varying directly with the distance from the axis. According to M. Gallois' explanation this relation would be an inverse one.

In a flowing stream these eddying movements are present in the forms described and in numerous intermediate states. Irregularities in the bottom sometimes send currents to the surface in the form of boils where there appears to be only a swift vertical velocity without much rotation. In the complex motion of the stream these visible disturbances are but the type of an infinite variety of similar movements taking place between small groups of molecules. These are not so evident to the eye, but consume in the aggregate a large proportion of the stream's energy. The asperities of the bed and banks send off these rotary groups with axes inclined at all angles and with velocities of translation having a resultant in the stream direction. Whatever resistance is offered along the air profile will manifest itself in complicating the direction of these movements of translation. The vital question is to know the direction of the resultant motion. Has it a component acting upward?

It is seen at once that vertical motion of the water itself can only be a local phenomenon. The resultant of all the external forces acting upon a given stretch of river will have a direction down stream, and will approach zero as the motion approaches uniformity. Its vertical component will be downward, not upward. The same thing will be true of all the complex motions in the interior of the liquid prism considered in the aggregate. It is true, however, that the reverse downward current need not equal in velocity the original upward one.[2] The diminution in velocity may be counterbalanced

[1] See *Engineering News*, March 23d, 1893.

[2] This fact is noted, without comment, by Mr. McMath in *Engineering News*, November 1st, 1879. It is thought that he was the first to suggest it in connection with suspension.

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by an increase in the moving area. This is an important point affecting the power of suspension. While there is no resultant vertical motion of the water, there is a resultant vertical thrust exercised upon sediment in the aggregate and acting as an efficient cause of suspension.

The thrust exercised by a current upon an immersed solid is

$$P = k\, F\, \gamma\, \frac{v^2}{2g}$$

and obviously varies with the square of the velocity. An eddy, having an upward component of velocity represented by 4, would exert an upward thrust, available for suspension, proportional to 16. In descending, the same water might cover twice the area, and have a downward velocity of 2. Then the corresponding downward thrust would be proportional to 8. The resultant of such a local system would be an upward thrust available for continuous suspension. In addition to this, particles carried by the vortex to the surface, and especially when freed at that point from its influence, would exhibit the phenomenon of temporary suspension, before gravity had again brought them back to the river bed. Cover the stream with systems of this character, and there results a modified form of what is actually seen. Eddies are induced by the rugosities met with in the earth and air profiles. The abrupt changes in the bottom, and its relatively unyielding character, cause the upward currents to have the greater velocity in general, though they must of necessity be restricted in relative extent.

The fact that the descending areas must be much greater than the ascending ones in order to give rise to this force shows that, in the major part of the stream, an active force is at work tending to carry material to the bottom, aside from the force of gravity acting upon the particles. This suggests, though it does not prove, the existence of another cause of suspension, and such a cause is actually found, as shown later.

Here, then, in obedience to the law which governs the thrust exercised by water impinging upon a solid, there is a resultant force acting upward in flowing streams, and capable of performing an efficient part in the continuous suspension of solid matter. While the aggregate of all the interior motions of the fluid can have no upward resultant, still the thrusts exercised upon solid suspended matter by these motions can and do have such a resultant as shown.

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It is evident that this force would disappear in circular conduits, flowing full, since a uniform profile may be assumed. The eddies would be thrown off with like intensity at all points toward the center.

M. Boussinesq[1] attributes increase in vortical motion at the walls to any one of these causes; increased mean local velocity, increased roughness of lining or increase in hydraulic mean radius. The effect of the first two causes is evident. The influence of the increase of area per unit of wetted perimeter is thought to be shown by the greater field of action allowed for oscillatory movements perpendicular to the walls. These vibrations help in detaching groups of molecules and cause such groups to suffer sudden changes in intensity of tangential friction at the walls which, in turn, favors their formation into vortices.

In a circular conduit running full this would indicate an increase in vortical movements at the walls with increase in diameter. In passing from the walls to the center of the current, M. Boussinesq thinks the vortical agitation should increase where the wetted perimeter is concave, as in the case of closed conduits of rectangular or rounded forms, running full, because the field of action for each vortex is constantly narrowing and interference increases. On the other hand, this agitation should be less and be practically uniform for a rectangular cross-section of indefinite length of base. The agitation should be of about the same intensity in a large circular or rectangular conduit running full as when running half full. The action of the free surface in reflecting these movements ought not to differ widely from the action of the upper half of the section upon the lower half when running full.

Considered as a whole, the vortex theory,[2] as advocated by Mr. McMath, is entitled to a position as one of the major causes of suspension. The statement that it is the sole cause, and that observation has never detected any other cause incident to the flow of streams which has produced these effects would seem too broad a claim.

(c.) *Theory Based upon Eddies and Relative Velocities.*—Partiot was probably the first to give prominence to the suspending power of

[1] "Theorie des Eaux Courantes," p. 47.

[2] A valuable paper by William Starling, M. Am. Soc. C. E., Chief Engineer of the Mississippi Levee Commission, is in press at the present time, which brings this theory prominently forward. The term "vortex theory" is here used in a restricted sense, as indicated by the context.

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eddies. He bases his explanation of suspension upon the presence, in these vortical movements, of exaggerated relative velocities which, in conjunction with the motion of the body, cause a thrust toward the most rapid filaments. The particles are then carried along with the motion of translation of the eddy.

Where eddies are not the acting force, he follows Dupuit's explanation as given hereafter.

(*d.*) *Theory Based upon Relative Velocities.*—This theory was brought forward by Dupuit. It utilizes the fact that all partially or wholly submerged bodies tend to move faster than the mean velocity of the displaced water. This makes a choice of path for relative motion necessary. The path chosen will be that which offers the least resistance, *i. e.*, diagonally toward the most rapid filaments.

Take the case of a floating homogeneous sphere. Let it have a velocity of translation in the stream direction equal to that of the mean of the displaced filaments. Assume uniform stream motion. Place it near the left-hand shore. The filaments of water at the right of the body are moving faster than its center of gravity, and by their friction tend to revolve it in an anti-clockwise direction. On the opposite side of the ball the friction is reversed in direction, and aids in producing the same rotation. What is the result upon transverse motion?

The ball will tend to place its center of gravity in the line of the resultant resistance of the filaments ahead of it. But in the theoretical case considered there is no resultant resistance upon the sphere, since it moves with the same mean velocity as the displaced water. The law stated above, then, does not call for any transverse motion under the conditions named.

Introduce the fact that the body has a velocity greater than that of the mean of the displaced filaments. A new element is introduced, for there is now a resultant resistance from the filaments ahead which is proportioned to the square of the relative velocity. This relative velocity, and consequently the resistance, increases until the resistance is equal to the accelerating force, when the motion becomes uniform. It is now incumbent upon the body to choose a path relatively to the fluid. It will tend to place its center of gravity in the line of action of the resultant resistance. This line of action is to the right of the center of gravity of the body. It will then take a diagonal path to the right, or toward the most rapid filaments.

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The line of application of the mean resistance will coincide with the line of action of the mean velocity of the water immediately ahead. At any point of its path the mean resistance offered will be least, *i. e.*, the velocity of the body will be most nearly equal to the mean velocity ahead, if the center of the body is in the line of action of this mean velocity.

In summing up, then, Dupuit's theory of suspension rests upon the fact that submerged bodies tend to move faster than the mean velocity of the displaced water, and in so doing choose the line of least resistance.

It is the only satisfactory explanation yet offered of the phenomena shown in the experiments with rotating vessels, which are described by M. Fargue and M. Gallois. It offers an explanation of the increase in suspending power, with increase in depth and velocity. In the "Report on the Mississippi River" by Humphreys and Abbot (page 254) it is shown that the parameter of the horizontal parabola of velocities 5 ft. below the surface, at any stage, is proportional to the square root of the corresponding mean velocity. The same should be true of the vertical curve.

The reasonableness and value of this theory seem evident. Its incompleteness as an explanation of the whole phenomenon is shown by the fact that it takes no account of the suspending power of eddies and offers no explanation of the presence of sediment above the line of maximum velocity.

Before stating the conclusions to which this analysis of theories leads, it will be well to place emphasis upon a cause of suspension which has only been indicated in a general way.

Dupuit[1] suggested that the resultant of all the pressures upon a submerged body was greater in flowing or agitated than in quiescent water. He explains it on the ground of relative velocities of the filaments as detailed above.

M. Flamant[2] has adopted the same idea as an explanation of the tendency of surface floats to move faster than the current. His argument is that the floating body weighs more in agitated water than the weight of the water displaced, and that the excess shows its presence in the increased velocity. The explanation implied is that there is a resultant upward thrust in the surrounding fluid due to its agitation.

[1] "Etudes sur Le Mouvement des Eaux," 1848, p. 217.
[2] "Hydraulique," 1891, p. 298.

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The explanation offered by Du Boys[1] for the increased velocity of surface floats has indicated the presence of a forward thrust, but has not called attention to the fact that this thrust has a direction above the horizontal. This follows from the discussion under (*b*) preceding. Experimental demonstration is wanting, but the fact that continuous suspension exists above the line of maximum velocity is an evidence that Nature shows the phenomenon indicated by theory.

It should be remembered that the line of maximum velocity has often been found at the surface. In such cases the suspending power due to the body's excess of velocity and tendency toward the line of least resistance acts from bed to surface in conjunction with the upward thrust due to the eddies. In the majority of cases it is found below the surface, and here a force must be acting downward to assist gravity in opposing suspension above the line of maximum velocity. This would indicate for such cases a decrease in suspended matter near the surface. The burden of the work then falls upon the upward thrust of the vortices. The fact that agitation increases from the line of maximum velocity to the surface tends to make the effect of this downward thrust disappear, since the latter decreases in intensity toward the vertex of the vertical curve. In this way marked differences in the intensity of sediment charge at the surface and at the line of maximum velocity are prevented. A difference in the amount carried at the surface ought to be shown according as the vertex of the curve of vertical velocities is raised or lowered—for instance, by the wind. The surface quantities should be greater when the maximum velocity is at the surface.

Conclusions.

It is believed that the suspension of sediment in flowing water may be attributed to three causes:

First.—The resultant upward thrust due to eddies, conditioned upon the facts that the earth profile offers more rugosities than the air profile and the effort exerted by a current upon a solid varies as the square of the relative velocity.

Second.—The resultant upward motion of solids due to the fact that an immersed body tends to move faster than the mean velocity of the

[1] "Sur la Marche des Bateaux," *Annales des Ponts et Chaussées*, January, 1886, pp. 199–242.

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displaced water and in such motion tends to follow the line of least resistance.

Third.—The viscosity of the water.

All of these causes will be present in every stream flowing under natural conditions. The first two causes will alternate in efficiency with the complex motion of the stream. At certain points of the vertical curves of velocities the second cause may entirely disappear as the curve becomes irregular, but such conditions are abnormal.

Experimental research is needed for further progress along these lines. Some of the more important questions awaiting investigation will be indicated.

Can it be shown experimentally that the power of suspension in flowing water increases with the increase of relative velocities in the vertical?

Can a measurable difference in volume of displacement be detected for the same body in quiescent and in flowing water?

What is the normal form of the vertical and horizontal sediment curve?

Can it be shown by experiment that suspending power increases with depth, the mean velocity remaining unchanged?

What is the influence of a contraction in width upon the form of the vertical curves of velocity and sediment?

What is the form of the vertical curve of velocities corresponding to incipient dragging for stream beds composed of the range of materials usually found in practice?

Can it be shown by experiment[1] that a heavier load of fine than of coarse particles can be carried with the same expenditure of stream energy?

What is the effect of temperature upon viscosity and mechanical suspension?

An apparatus has been designed for the purpose of studying the first of these questions. It consists of a long, narrow trough, fitted with glass sides for nearly its whole length, which is joined to a wooden receiving reservoir. A steady flow is set up from the reservoir through the experimental channel.

Two submerged jets, each covering the entire width of the channel,

[1] A general method of experiment has been indicated by Ashbel Welsh, Past-President Am. Soc. C. E., *Transactions*, American Society of Civil Engineers, Vol. xi, p. 162.

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are introduced near the reservoir through suitable copper guides, arranged so as to create the minimum amount of agitation in the jets and in the steady flow progressing independently.

These jets discharge under a head of 40 ft. One is placed near the bed, another near the surface of the channel. The object sought in the design was to obtain a means of controlling the form of the vertical velocity curves by varying the discharge from each of the jets. This is controlled by stop-cocks. The velocities at different parts of the vertical are measured by a cluster of Pitot's tubes.

The experiments have not yet been made. Preliminary trials have shown, however, that the control of the curve of velocities is appreciable for a sufficient distance from the jets to afford ample field for experimentation.

The author takes pleasure in expressing his deep sense of obligation to Prof. E. A. Fuertes for constant assistance and suggestions, and to Prof. I. P. Church for kindly criticism, which has led to modifications in some of the views expressed. This opportunity is gladly taken to mention the kindness of M. Edouard Collignon, Inspecteur Général des Ponts et Chaussées, in affording, among other courtesies, unusual library privileges at the École Nationale des Ponts et Chaussées, Paris; and of Prof. Conrad Zschokke, of Zurich, in assistance rendered in various ways. He wishes, also, to express his appreciation of the continued courtesy shown him by M. Cordier, the librarian of the Ecole des Ponts et Chaussées at Paris, and by Prof. Rudio, the librarian of the Zurich Polytechnikum, during his use of those libraries.

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DISCUSSION.

T. C. CLARKE, M. Am. Soc. C. E.—It is rather surprising to notice in the paper the statement that no measurements have ever established the fact that there may be a convexity in the center of a stream. It is the opinion of pilots on the Mississippi River, who observe such matters very closely, that a rising river can be distinguished from a falling river by the fact that on a rising river the driftwood floats along the shores, while on a falling river it floats in the center of the stream. The belief is that a rising river is convex and the driftwood floats away to the sides, and that a falling river is concave and the driftwood comes to the center. Mr. Clarke.

L. L. BUCK, M. Am. Soc. C. E —In the rapids of the Niagara River, below the falls and above the whirlpool, the water at the center of the stream is about 10 or 12 ft. above the level at the sides, the height being greatest where the stream has the highest velocity. At one part of the whirlpool there is also an elevation of the surface of perhaps more than a foot, due to the fact that two currents at different depths pass at this place and force the water upwards. Mr. Buck.

A possible explanation of the curvature of the surface of the Mississippi River is that the slope of a river in a given distance is less during falling than during rising stages. If it is true that the slope in a given mile causes the piling up of the water in midstream, owing to the greater velocity there, the diminished slope during a falling stage of the river would account for the concave surface at such time.

FOSTER CROWELL, M. Am. Soc. C. E.—The explanation of the convexity of surface of a rising river may be that at such a time the entire section of the river is rising throughout a long reach, and the accumulation of water will naturally take the path of least resistance. Consequently, it will have an opportunity to concentrate farthest from the banks, which will exercise a retarding influence on the flow. When the river is falling, the principle of least resistance will explain the concave surface. There, of course, can be no movement of free water without a surface slope in its direction. Mr. Crowell.

During a large flood in the Chemung River the driftwood, instead of passing down the river, left the center and moved up stream at the sides; in other words, eddies were formed along the banks. The fact that the driftwood did proceed up the river was made a great deal of by some parties who were plaintiffs in suits brought to recover damages from a railroad company owning a bridge which was claimed by them to have increased their losses. Although the bridge with all the country about it was submerged, the fact that driftwood was seen to pass up the stream along the banks was offered as positive proof that it was the bridge that had dammed the whole river and turned the current up stream. In this case the driftwood must have moved along the line of least resistance, and it may be assumed, therefore, that the surface of the water in the center of the river was higher than at the sides.

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CORRESPONDENCE.

M. Partiot. M. LEON PARTIOT.*—The lateral and vertical currents in a river are generally determined by the condition of the bed of the stream, which is, in turn, subject to their action. An intimate relation connects the width, slope and depth of a river with the velocity and movement of the water. It is to be hoped that the author's interesting researches will be continued so as to carry as far as possible toward completion the study of the reciprocal action of currents and river beds. New and useful information concerning the controlling laws will probably be found in this way.

A sudden increase of the width leads to a corresponding shoaling. The water which is flowing upward is then lifted by the slope on which it rises, and, if the upper currents are not swift, it reaches the surface, where its presence is indicated by a special motion. The appearance of the surface often indicates the nature of the bottom in this way during calms, and allows boatmen to shape the course they should follow.

Other facts of the same nature may be observed, such as transverse currents in the bends of a stream. The water which is directed by the upper part of the bed against the concave bank strikes that bank and rises slightly. Its head reacts on the lower liquid strata and produces, with them, a descending current. This current, in connection with that of the lower strata, attacks the concave bank, and the eroded material is transported down stream and toward the bottom. There is thus formed in the body of the water a current which follows a more or less helicoidal curve down stream.

The solid particles which it carries are deposited upon the convex bank and form a shoal below the point corresponding to the apex of the curve of the concave bank. This shoal decreases the width of the river, and thrusts the water toward the prolongation of the latter bank, so that the deep part due to the concave curve is prolonged down stream a certain distance from the apex of that curve. The mean and uniform depth suited to the width of the stream is only produced in a cross-section below this shoal.

The formula—

$$P = m\frac{qV^2}{Rg}$$

which the writer has given† to express the eroding force, P, of the water in a concave curve makes this force vary as the square of the velocity, V, of the current and inversely as the radius of curvature, R, of the bank. When a longitudinal dike is constructed along a concave bank, the friction of the water against the dike diminishes the velocity, V; in order that the eroding force, P, and, consequently, the depth along the dike, may be maintained, the radius of curvature of the dike should decrease inversely as the square of the velocity. This rule can often be applied, although not always.

* Inspector-General, Ponts et Chaussées, Paris.

† Mémoire sur les Sables de la Loire, p. 27, Paris, 1871.

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M. A. FLAMANT.*—The author is to be congratulated on the remarkable manner in which he has analyzed and summarized the numerous and varied works produced on the important subject of the suspension of solids in flowing water, which, nevertheless, is far from being completely elucidated. The writer agrees entirely with the author's statement that new experiments are needed, and has little to say, except that the ninth conclusion, on page 314, is apparently expressed in terms exceeding the author's meaning. M. Flamant.

"9. The weight of suspended material per cubic unit of water decreases from rise to embouchure."

The source of rivers is generally clear and limpid, and the quantity of suspended matter increases at first.

In order to express the proposition in an exact manner it would be necessary to take into account, at each point of a stream, the state of the banks and bed, and compare the weight of the matter previously in suspension and deposited there by the water on account of a diminution in velocity with the weight of the matter eroded by the current from the banks and bed and taken in suspension at the point considered. The amount of matter in suspension per unit volume of water increases or decreases according as the second of these two weights is greater or less than the first.

The two phenomena of deposition and erosion may very well occur simultaneously at the same point or at closely neighboring points of the same stream, and the writer does not at all believe that the materials from the banks in the higher portions of a river continue their course, without interruption, to the sea.

The writer considers it also necessary to take account of the greater or less facility with which the suspended materials may be divided.

In the case of the sands of the Loire, for example, the entraining current is insufficient to divide them. At the most, it gradually reduces their dimensions by mutual friction, but does not strike one particle against another with sufficient force to break them into several pieces. It may thus be said that, so far as the entraining current is concerned, the particles are indivisible, and, consequently, the quantity of suspended material diminishes proportionately to the diminution in velocity.

But it is otherwise when the suspended or entrained material consists of clay or mud, which may be regarded as indefinitely divisible, or having a condition of division increased by the least agitation of the liquid containing it. There will then be no tendency for this material to be deposited, and if at the same time the river bed is muddy, it is not impossible that under the action of the current a part of the mud of the bottom and banks may be taken into suspension.

*Inspector-General, Ponts et Chaussées, Algiers.

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M. Flamant. It is probably in differences of this nature that an explanation should be sought for the variation of opinion, mentioned on page 280, between M. Fargue and other observers.

There are, then, in all streams portions in which the proportion of suspended material is increasing, and others in which it is diminishing, according to the character, volume and weight of the suspended material, and especially the readiness with which it may be divided, and also according to the nature of the banks and bottom. If the bed cannot be washed away, and the suspended materials are indivisible by the current, it is certain that the quantity of transported material can only diminish as the velocity of the current falls, and consequently the transporting power of the water decreases.

Aside from the materials in suspension, properly speaking, that is to say, those entirely surrounded and supported by the water, it is necessary to take into account in a stream the material entrained by the current which rolls on the bottom and rests at one or more points against the adjoining particles, either moving or at rest. The distinction between material entrained in this manner and that in suspension is not always drawn clearly, and the writer considers it very difficult to establish. It may frequently happen that a grain of sand, for example, which has rolled for a time along the bottom, becomes suspended momentarily by an eddy. It may also happen that a grain of sand or a pebble wears away after rolling a certain time, and, without becoming broken, acquires dimensions sufficiently small to permit it to be taken into suspension. An increase of suspended material may thus take place in the direction of the current. This phenomenon is not hypothetical, for it occurs in several streams, especially in the Rhone, which first carries pebbles rolling on the bottom, and, afterward, towards its mouth, is charged with suspended materials formed by the wearing away of the pebbles, which decrease in size rapidly as they move down stream.

The writer thinks the author's ninth proposition should be modified in that it is too absolute.

Mr. Starling. WILLIAM STARLING, M. Am. Soc. C. E.—This excellent paper contains a nearly complete summary of the existing state of knowledge in regard to the transportation of solid matter by flowing water. The bibliographical part is extremely valuable and evidently represents the result of a vast deal of research. It is a repository of what is of real worth and originality in the literature of this interesting subject.

To those engineers who are concerned with the improvement of sedimentary rivers, the subject of this paper is of the highest importance. A few years ago the whole plan of the improvement of the Mississippi River was made to depend upon the construction to be placed upon the laws governing the suspension of the silt in the stream. Upon these laws rests the decision which must be pronounced as to

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the proper means of protection of the alluvial lands from overflow— whether by levees, outlets, cut-offs or what not—and also as to the best means of securing the banks from erosion. To such engineers and to the whole profession it is therefore a great service rendered to embody in one easily accessible paper the results of the researches and experiments which have been made during the last hundred years or so, and which have heretofore been widely scattered in treatises, transactions and pamphlets, in various languages, places and periods. Mr. Starling.

The paper, however, is far more than a mere bibliography. It gives a fair summary of the contents of each of the authorities mentioned, and enables the student who wishes to go deeply into the investigation of the subject to discriminate between what will probably be of use to him and what will be comparatively valueless. The second and third parts of the paper, the "Discussion of Observed Data" and the "Discussion of Theories of Suspension," are very interesting, and the evidence is very judiciously and ably summed up.

The list of authorities might probably be enlarged a little. In Richthofen's "Führer für Forschungsreisende,"* there is an excellent chapter on the mechanical work of flowing water, which contains a good summary of the knowledge and doctrine of that subject. Though founded on the observations of others, it almost rises to the dignity of an original work. The valuable paper of Dr. W. H. Brewer, in the "Memoirs of the National Academy of Sciences" for 1884, mentioned by Richthofen and others, on the influence of salts and foreign matters in solution on suspending power, has apparently escaped the author's notice. There were extensive observations made at Fulton, Tenn., on the Mississippi River, by Mr. W. G. Powless, for almost a whole year, in 1879–80.† These contain valuable data relative to the quantities of sediment carried at different stages and under varying circumstances, and its distribution in vertical planes, as well as studies of the movement of sand waves, which rank among the most nearly complete of their kind. The observations made at Carrollton, in 1879–80, under the direction of Captain Smith S. Leach, and thoroughly discussed by him,‡ and the report of Arthur Hider, M. Am. Soc. C. E., on the Lake Providence Reach,§ are evidently known to the author, but seem hardly to have been accorded that place which is due to their merit.

The paper of B. M. Harrod, M. Am. Soc. C. E., on the protection of banks, published in the "Report of the Mississippi River Commission" for 1885,‖ contains an excellent summary of the sediment

* Pages 133–208.

† In the "Report of the Mississippi River Commission" for 1881, pp. 66–121.

‡ "Report of the Mississippi River Commission" for 1882, pp. 98–115.

§ "Report of the Mississippi River Commission" for 1882, pp. 80–98.

‖ Afterwards printed, in a modified form, as a separate pamphlet, "On the Caving Banks of the Mississippi River." New Orleans, 1886.

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Mr. Starling. observations made on the Mississippi up to that date, with comments and deductions of much value.

The voluminous work of Mr. J. G. W. Fijnje, in the Dutch language, entitled "Considerations on Certain Rivers, Including Those of the Netherlands,"* contains the records of some valuable investigations made on the lower Rhine, under the auspices of the Dutch government, and a thorough discussion of them, as well as an elaborate examination of the data derived from foreign experience, with particular reference to the relation between stage of river and quantity of sediment carried, and to the proportions of the latter in the different reaches of the stream, upper and lower. There are also the record of experiments on the influence of salt water on the subsidence of suspended matter, and observations on the distribution of sediment in vertical and horizontal planes.

Some interesting and important "Observations upon the Erosion in the Hydrographic Basin of the Arkansas River,"† were published by John Casper Branner, Ph.D. (then State Geologist of Arkansas), in 1893. These observations contained, among other things, 179 determinations of matter carried in suspension, and a smaller number of determinations of matter carried in solution. They were taken at Little Rock during the year 1887–88. As the pamphlet was published in a very limited edition, some extracts from it are given:

"The matter in suspension is greatest during a sudden high rise; but after the water in the stream stands at any high mark for a few days, the decrease of the amount of suspended matter it carries is very marked. The amount of sediment carried by the river varies widely also with the same gauge-reading at any stage, being greater with a rising and less with a falling river.

"The greatest amount of mechanical sediment found in the water during the year under consideration was 225 grains to the gallon ($= \frac{1}{27.9}$) when the river stood at 17 ft. on the gauge, and after protracted rains (extreme high water at Little Rock is about 28 ft.). It should be added, however, that while this high water may be taken as a type of the ordinary rises, there are times when there is but little or no rise, no increase in the volume of water discharged, but a very marked increase in the amount of mechanically suspended matter. In October, 1891, occurred one of these so-called 'red rises' of the Arkansas River, and although the river was quite low—marking only 3.9 ft. on the gauge—it carried 761 grains of matter to the gallon, of which only 46 grains was matter in solution (that is, the matter in suspension was $\frac{1}{15}$, by weight).

"The matter in solution bears no constant relation to the volume of water, though in a very general way it varies inversely with the volume of the water, and ranges from 11 to 70 grains to the United States gallon. The amount carried down, in this form, from October, 1887, to September, 1888, was 6 828 350 tons. During the single month of May, 1888, 1 161 160 tons were carried out in solution. Taking the observa-

* "Beschouwingen over eenige Rivieren, waaronder Nederlandsche," 's Gravenhage 1884–1888, 3 vols. of text and 1 of appendices. See Vol. iii, pp. 876–904.

† Reprinted from the "Wilder Quarter Century Book," Ithaca, New York, 1893.

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tions for the entire year under consideration, the matter in solution is equal to about 0.31 of that in suspension. These relations, however, are not constant. In November, 1887, for example, the dissolved matter was more than six times as much as the suspended matter—while on October 13th, 1891, the suspended matter was more than thirteen times the matter in solution." Mr. Starling.

The paper of Mr. Carl Barus, to which the author has referred in a foot-note, "On the Subsidence of Fine Particles in Liquids,"* deals merely with still water, and is not concerned with the relations between suspension and velocity. Mr. Barus shows, after Maxwell, that a drop of water whose diameter (0.025 mm.) is about one-half that of a human hair would descend in air at a rate of only 2 cm. per second. Hence a drop, the dimensions of which lie near the limit of microscopic vision, say 0.00025 mm., is practically stationary, its rate being 0.002 mm. per second. Mr. Barus thinks the suspension in water, for an indefinite time, of solid particles, is attributable partly to molecular action, partly to viscosity, partly to chemical relations between the solid and the fluid. It is found that the rate of subsidence increases rapidly with the temperature, being "enormously more rapid at 100° than at 0°." The viscosity of water at 100° is only about one-sixth of its value at zero. It is thought, however, that this accounts for only a small part of the difference in suspending power.

Mr. Barus's paper contains the results of an elaborate series of experiments on the effect of precipitants. He shows that "acids, salts, alkalies, indeed foreign material in general, when added to distilled water permanently turbid with some finely comminuted insoluble solid, in quantities not too large, increase the rate of subsidence in a marked degree, in numerous cases even many hundred-fold." It has long been known that the precipitation of sediment is much more rapid in sea water than in fresh water, and the fact has an important bearing on the formation of bars at the mouths of rivers. The paper is a very interesting one, and is well worthy of perusal.

The writer will confine his remarks mostly to the lessons which have been taught by recent experience on the Mississippi.

Reference is made† to the law of decrease of mean velocity from the rise to the embouchure of rivers. It will not be disputed that the torrential part of a stream, in time of flood, has a mean velocity greater than that of the fluvial portion; but after a river has become a river, the law, if true at all, must be understood with many reservations and exceptions. The mean velocity of the Ohio, for instance, or of the upper Mississippi, is less than that of the lower Mississippi, of which they are principal tributaries. In the flood of 1881, which was a great one in the upper Mississippi, the maximum velocity at Prescott, just below St. Paul, as determined by actual observation, was about 3 ft.

* Bulletin No. 36 of the United States Geological Survey.
† On page 280 of the paper.

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Mr. Starling. per second; at Winona, 80 miles below, about 3.4 ft.; at Clayton, 100 miles below Winona, about 4 ft.; at Hannibal, about 300 miles below Clayton, about 5.2 ft.; at Grafton, about 90 miles below Hannibal, about 4.6 ft.; at St. Louis, 35 miles further down, after the reception of the Missouri, 6.8 ft. In the lower Mississippi, from Cairo to Carrollton, it was probably about 5.5 ft.

This statement is very commonly made, even in works of authority, and, no doubt, it is often true; but it is often assumed, from the well-known fact that the slope of streams generally decreases, more or less regularly, from the sources to the mouth. Calculations made by means of the ordinary formulas will therefore generally give a constantly decreasing velocity. Observations on the Mississippi, however, show a great increase of the coefficients of the formulas as the mouth of the river is approached; or, in other words, as the slope grows flatter. Not only so, but the increased volume derived from tributaries and increased drainage area often compensates, or more than compensates, for the flatter slope in the lower reaches, while in the alluvial portions the shape of the cross-section, and sometimes the improved form of the trace of the stream, is conducive to greater velocity as the mouth is neared.

The topic of greatest interest in the paper is, of course, the connection between transporting power and velocity, and this important subject receives full treatment. It is thought, however, that undue consequence is attributed to the theory of Dupuit, by which suspending power is ascribed to the difference of velocity between adjacent fillets or, rather, layers.

This explanation satisfied Humphreys and Abbot, Debauve and other engineers of reputation. The wonder is that it satisfied M. Dupuit himself, who was by no means a mere theorist or formula fiend, but a man of eminently sound common sense and of much experience. The truth is that Dupuit's theory is not at all adequate when submitted to a quantitative test to explain the observed phenomena.

To take an instance from actual practice. The average quantity of sediment carried by the Mississippi River is about $\frac{1}{3000}$ by bulk of the water. The average depth of the lower river at high water may be called about 60 ft. The maximum velocity may be taken at 6 ft., the bottom velocity at 3 ft. This may be regarded as near the extreme limit. The diameter of the largest particles carried in suspension will be taken at 0.01 ft. For the purpose of the investigation, they will be considered as cubical in form.

If the place of maximum velocity in the vertical be taken at one-third of the depth from the top, there will be a variation of 3 ft. per second in velocity in 40 ft. of depth; that is, between the upper and lower sides of a cube of an edge of 0.01 ft. there will be, at the same rate, a difference of velocity of $\frac{3}{4000}$ or 0.00075 ft. per second.

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Mr. Starling.

Now, a solid body of a specific gravity of 2 when immersed in water will lose one-half of its weight, as the expression is, that is, it will require a force equal to the weight of a cube of water of the same size to hold it up. The pressure of running water on the bottom face of the cube is equal to the weight of a column of water whose base is the pressed surface and whose altitude is the head, due to the velocity, multiplied by a coefficient which may be taken, on the whole, as 1. Now, if a be the edge of the cube, h the height due to the velocity v, and γ the weight of a cubic foot of water, there will result the equation—

$$\gamma a^3 = \gamma a^2 h ;$$

that is, for a cube of the specific gravity 2, the height is equal to the edge of the cube; in the present instance 0.01 ft. The velocity due to this head is 0.8 ft.

This velocity, therefore, is required to hold up the given cube. But the difference of velocity between the upper and lower sides of the cube, as seen above, is 0.00075 ft., or only 0.001 of that required.

The smallest particles suspended in the water of the Mississippi are estimated to be about $\frac{1}{3000}$ in., or say, for convenience, 0.00003 ft. in diameter. The velocity due to this height is about 0.044 ft. The difference between the velocities at the upper and lower sides of such a cube is only $\frac{1}{444444}$, or 0.00000225; that is, about $\frac{1}{30000}$ of that required.

Instead of magnitudes derived from theory, the results given by experiment, as tabulated by the author from M. Thoulet,* may be taken. To bear up a spherical particle of specific gravity 2, of a diameter of 5 mm., a velocity of 156 mm. per second was required. The difference of velocities between the upper and lower sides of the sphere (of 0.0164 ft. diameter) is 0.00123 ft. or 0.37 mm., only $\frac{1}{422}$ of that required for suspension. A particle 0.2 mm. in diameter required a vertical velocity of 23 mm. The difference in velocities for such a body is about 0.00005 ft., or 0.015 mm., only $\frac{1}{1533}$ of that required. In this case, as in the theoretical illustration, the smaller the body, the less adequate is the explanation.

It must be remembered, too, that in Dupuit's theory the difference of velocities is not all in a vertical direction. Indeed, Dupuit has left it undetermined how much of the difference is effective for suspension. It has been assumed, in the discussion, to be all effective, in order to show the inadequacy of the theory in a striking manner.

There is a far more remarkable phenomenon than the transportation of silt by running water going on every day, and that is the suspension of solid or liquid bodies in the air. A beam of light shining through a crevice in a dark room reveals millions of small particles of perceptible and measurable size floating under the influence of gentle and ap-

* See p. 312.

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Mr. Starling. parently capricious currents. Probably the greater part of these minute bodies have a specific gravity 1 000 or 1 500 times greater than that of the air in which they are suspended. Flowing water is known to be actuated by a vast number of internal currents or vortices, which, as Boussinesq has shown, consume by far the greater part of the energy of the stream. The more violent of these are plainly visible in every running water-course; and in powerful rivers, they form very marked phenomena, constituting eddies and boils which are of constant recurrence and are of great vigor. The Mississippi, to the eye, appears to be little more than a succession of such whirls. In situations where excessive disturbance of the bottom is taking place, as at crevasses in levees, the surface movements are very violent and are, indeed, a faithful index of the commotion going on below. A submerged reef or wreck, though at the depth of many feet (20 or 30, perhaps more), is marked, to the acute eye of the pilot, by a "break," as he calls it, on the surface of the river.

While it must be conceded that upward vertical movements of the particles of water will be compensated, in the long run, by downward movements of equal extent, yet, as has been observed by Mr. McMath, quoted by the author, the areas and velocities of such movements may be very different. Suppose, for instance, in a given locality, an equal number of particles to be ascending and descending; but suppose the velocity of ascent to be double that of descent, and the area occupied by the particles, consequently, only half; then the total upward thrust exercised by the current would be twice the downward thrust due to the same agency.

Captain Leach has very judiciously observed* that a distinction is to be drawn between sediment permanently and intermittently in suspension. On this discrimination hinges a very important part of the question. The lighter portion of the sediment, the fine clay or loam, may be carried almost by the mere viscosity of the water. The sand may be transported only a few miles, taken from one side of the river, added to the other, without affecting the general cross-section.† At all stages, there are areas of erosion and areas of deposition; but these areas are not the same for different stages; rather, they alternate, and the area of erosion for low water often or generally becomes the area of deposition for high.

It is possible, then, that the sediment intermittently carried from bar to pool and from pool to bar may be alternately lifted by upward currents and dropped by downward. There may, therefore, very well be a predominance of upward movement at one place and of down-

* "Report of the Mississippi River Commission" for 1882, p. 105.

† See a very interesting and able paper by J. B. Johnson, M. Am. Soc. C. E., entitled "Three Problems in River Physics," published in the *Proceedings* of the American Association for the Advancement of Science, Vol. xxxiii; also separately printed at Salem, Mass., 1885.

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ward movement at another. The load of sediment intermittently carried is very great, and the material, for the most part, heavy and coarse. The alternate scour and fill at the different discharge stations on the Mississippi River usually amounts to 25 000 or 30 000 sq. ft. of cross-section annually, in a total area of 200 000 or 250 000 sq. ft. Mr. Starling.

There is still a quantity permanently suspended. This is probably light silt, either brought down by the tributaries, especially the Missouri, or picked up from the banks in exchange for heavier matter brought from above and dropped. To transport this finely divided earth is a comparatively easy task. Indeed, it might almost be carried from Cairo to New Orleans in the time required for subsidence in still water. General Comstock has found that clay, put in a glass tube and hung up with water, took three weeks to subside.

There is a great variation in the testimony of very competent observers as to the relative proportions of sediment carried in suspension and rolled along the bottom. These discrepancies were to be expected, considering the great differences in places, times and conditions which have prevailed. A very remarkable difference of opinion, however, is manifested in the reports of Arthur Hider, M. Am. Soc. C. E., and Captain Smith S. Leach, in 1880. The observations of the former were conducted at Lake Providence, of the latter at Carrollton. The two stations are 415 miles apart. The latter is near the mouth of the river. Mr. Hider thought the proportion of sediment rolled to that suspended about 10 per cent. Captain Leach thought that 0.8% was a full estimate.

It may very well be that both these engineers were right. There is no reason whatever why the relative proportions of the burden borne at the two stations should be the same. The materials transported are very different. At Lake Providence they consist largely of coarse sand and gravel; at Carrollton probably, for the most part, of light sand and clay. At Fulton, Mr. Powless thought the proportion about 2.5 per cent. His estimates were made, however, from the movements of the sand reefs, and these, in Mr. Hider's opinion, constitute but a small part of the total movement, large quantities being swept along close to the bottom and not detected in sounding.

It is probable that low-water years, on the whole, are periods of deposition, and high-water years periods of scour. At least, this seems to be the conclusion to which observations on the Mississippi point. If this opinion be confirmed, it will lead to the inference that silt-bearing streams have at some times insufficient power to carry to the sea the burdens of sediment derived from their tributaries; and that at others they have a superfluous amount of energy, so as to enlarge their beds. It will be evident, then, that great caution should be used in drawing conclusions from a small number of observations, or those covering limited periods of time.

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Mr. Starling. On page 300 of the paper is given a table, indicating the distribution of sediment in a vertical sense. To the figures there set forth may be added the results of Mr. Powless's observations at Fulton, Tenn., in 1879–80. This engineer found the distribution to be as follows: Surface, 27.5%; mid-depth, 36; bottom, 36.5.

The table on page 310, showing the dimensions, movement, etc., of sand waves, may also be extended a little from the same authority. At Fulton: Mean velocity, 5.8; average depth, 45—60; height of waves (average), 4.7; length, 600; movement of crest per day, 22. At Bullerton: Velocity, 3.5; depth, 10—30; height of wave, 4.4; length, 300; movement of crest, 13; observer, W. G. Powless; date, November, 1879—November, 1880; authority, Report of Chief of Engineers for 1881.

The author is entitled to the thanks of the profession for the learning, industry and ability displayed in presenting to them so much valuable information; and to those of this Society in particular for enriching its *Transactions* with so excellent a paper.

Mr. Wisner. GEORGE Y. WISNER, M. Am. Soc. C. E.—The author has placed before the Society in his interesting and valuable paper nearly all the available data relative to the transporting power of water in motion, and plainly states the hydraulic laws governing such action. In his conclusions he attributes the suspension of sediment in flowing water to three causes, viz., the resultant upward thrust due to eddies, the resultant upward motion of solids due to the fact that an immersed body tends to move faster than the mean velocity of the displaced water, and the viscosity of the water. To these three causes should also be added a fourth, due to change of curvature of the river bed, which in alluvial streams is the principal factor tending to produce unequal depths in the channel. The velocity of the current for the upper two-thirds of the depth, being much greater than that of the bottom stratum, forces the main volume of the water against the concave bank and causes a decided inclination of the surface towards the convex bank; and since this movement is very light near the river bottom, the difference of level produces a transverse pressure on the particles of the river bed, which may be many times that due to the normal current in a straight reach of the river. The molecules of water, after impinging along the concave bank, will be forced down by the pressure diagonally towards the bottom, and thence across the course of the threads of the current along the river bed, thus forming a system of converging forces, the resultants of which constitute the eroding forces which tear up particles of the bed near the concave bank and deposit them in the slack water along the opposite shore. The maximum effect of these converging forces occurs at times of high water, producing over-saturation in curves, and causing bar formation in straight reaches immediately below. At times of low water these deep pools are practically level, and the river slope is concentrated in

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the short reaches across bars through which low-water channels are eroded, giving rise to the well-known anomaly of bar formation at high stage of alluvial rivers and channel deepening at low stage. Mr. Wisner.

At crossings between curves a cross-section normal to the direction of flow is usually much wider than a mean width of channel; and, the transporting power of the river consequently being less, a deposit takes place.

Aside from the effect of eddies and curves, the relative velocities of the strata at different depths create the force tending to hold matter in suspension, and since relative velocities for two streams having different depths and the same mean velocities will be less in the deeper stream, the shallow stream will have the greater eroding power.

A particle of sediment in suspension having a greater density than water will undoubtedly be subject to an accelerating force tending to move it faster than the current, equal to the specific weight of the particle in the fluid multiplied by the sine of the angle of the slope. The action of the relative velocities of successive strata and of the accelerating force due to gravity will tend to lift the particle until the plane of maximum current velocity is reached, above which the effect of the relative velocities will be negative. Consequently a greater amount of sediment should not be found at the surface than at three-tenths of the depth below the surface, as stated by Humphreys and Abbot in their Report on the Mississippi River.

A revolving ball thrown horizontally in the air, or a flat stone thrown along the surface of still water, will curve either to the right or left according to the direction of revolution. Owing to the viscosity of water the slower revolving motion of a particle in suspension, due to the unequal velocity of current on opposite sides, will for similar reasons give an upward curve to its path through the water in which it is suspended until it is lifted to the plane of maximum velocity, above which the velocities decrease towards the surface and produce a downward pressure on particles brought to the surface by eddies.

Since a certain amount of force is expended in lifting a particle from the bottom, the effect is to reduce the relative velocities of the strata through which it passes, and since the mean velocity of the stream is a function of these relative velocities, it is evident that the mean velocity of a stream for a given stage must decrease with the load carried in suspension.

For any given mean velocity of flow a shallow stream requires a much greater slope than a deep river, and as steep slopes give strong relative velocities from the bottom upwards, the transporting power of a river is much greater on the upper tributaries than near the mouth. For the same reason, the amount of sediment which a stream will transport, for any given stage when rising, is much greater than

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Mr. Wisner. for the same depth when the river is falling, since the slope is always greater in front of the flood crest.

Airy and Law show theoretically that the eroding power of a stream varies as the sixth power of the bottom velocity, but it must not be inferred from this that the eroding force and transporting power vary as the sixth power of the mean velocity of the river, as has been stated in recent works on this subject. It is quite certain that the bottom velocity of a stream does not increase in the same ratio as the mean velocity, and it is probable that bottom velocities are functions of the roughness of the river bed and of the relative velocity of current from the bottom upward, or the river slope, since the relative velocities depend on slope. Airy's formula also assumes a direct impact of the current on the particle, which is seldom the case, and since the thread of the current diverted from its path by a grain of sediment on the bottom loses but little momentum, it is very probable that the actual effect is much less than the formula would indicate.

The loss of force from impact varies with the depths of the stream, and consequently a given current velocity might produce scour in a shallow stream and have no effect whatever in deeper channels. It is therefore evident that Airy's formula and Dubuat's velocities at which different sizes of gravel will commence to move in currents have but little value in practice.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E.—Airy's fundamental law that the transporting power of flowing water varies as the sixth power of the velocity is as true and immutable as Torricelli's general law in hydraulics that v varies as $\sqrt{2gh}$. It is curious to note how many practical men still dispute the truth of Airy's law, but the facts which they present to substantiate their views and conclusions show but too plainly that they are laboring under a misapprehension of the first magnitude.

Nearly all the difficulty in comprehending Airy's law can be overcome entirely by simply noticing the fact that this law bears the same relation to the transporting power of water that Torricelli's law, v varies as $\sqrt{2gh}$, bears to practical engineering formulas now in use for the flow of water. It is simply a general law, and consequently is of little or no practical value at the present time, for the obvious reason that the influence of friction, viscosity, and all the other unknown disturbing causes, is yet to be introduced. When discovered and understood, these factors will some day modify the general law. In the present state of the knowledge of this subject, for any one to try a simple experiment and then proclaim Airy's law all wrong, is on a par with the procedure of a young engineer who not long ago tried an experiment with a water supply pipe line, and made the discovery that the velocity of discharge did not equal $\sqrt{2gh}$, and therefore Torricelli's law was all wrong.

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Mr. Le Conte.

It is quite clear that Airy's law will have to be supplemented by a large amount of experimental research as to modifying coefficients before it will be in such a shape as to be of much practical use. Here is an opportunity for some young ambitious member to render a great public service. When such experiments are completed, engineers will no doubt have many useful transporting-power formulas, which will certainly rival Kutter's in the number and complexity of their sub-coefficients.

The author calls attention to the wide difference in opinion among river engineers as to the ratio of material carried in suspension and that moving along the bottom. This can all be explained if the character of the stream in each case be fully considered. The writer's experience has been largely confined to rivers similar to the Missouri, and naturally his impressions accord closely with those of the Missouri River engineers. He has the best of reasons to know that in all such sedimentary streams the larger half of the material transported during the year is not carried in suspension. During flood stages especially, the entire sandy bed of the river, from bank to bank, and to a depth of 10 to 12 ft., is moving bodily down stream like an Alpine glacier. The sand waves on top are of no importance compared with the great bed-flow of sand *en masse*. Moreover, where the flood-slope is 4 ins. per mile, the transporting power of this bed-flow is such that coarse shingle and cobblestones are transported bodily down stream, corresponding in size and weight to those transported by water in the upper river, where the flood slope is 36 ins. per mile.

The author mentions a few experiments made by the writer in 1879 giving the variation in percentage of sediment in suspension in a vertical. These limited observations were taken with a certain object in view, and at a certain stage of the river, about the middle of the front slope of the flood wave, and the writer thinks the observed maximum at mid-depth is largely due to a transient condition at this stage. It may be safely stated that as a rule the weight of sediment in suspension increases more rapidly than the depth from the surface to the bottom.

As to the distribution in a horizontal line the data collected by the Missouri and Mississippi River Commissions, both agree that the percentage in suspension is practically constant all the way across the river. As to the most potent cause of suspension, the writer thinks M. Partiot is undoubtedly correct. The writer has had rare opportunities for studying the eddying motion of particles in suspension, from the point where they first leave the bottom until they reach the surface above. This upward inclined eddy motion does not in anyway conflict with the general horizontal flow of the stream. The whole motion can be perfectly understood in all its confusing details by simply comparing it with the identical phenomena observed

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Mr. LeConte. at an ordinary camp-fire in the open plain, the smoke, wafted by a gentle breeze, rising upward in the form of an inclined eddy, expanding as it rises. This likeness in the two cases is so striking than an observer watching the natural operations of suspension could well imagine himself in a balloon looking down upon a multitude of tiny camp-fires, and a gentle uniform breeze wafting the myriads of rising columns of smoke, so that they all incline in one direction. All the complex motions in the two cases are in every respect identical. A slight irregularity in the bottom is all that is necessary to start and maintain an eddy.

The author touches lightly on the subject of viscosity. The writer has given much attention to studying the effects of viscosity on the transporting power of running water. The phenomena are fully developed in hydraulic dredging operations on river and harbor works, the column of water discharged from the large pumps being charged with all percentages and every class of material, some in suspension and some rolling along the bottom. The results show conclusively the powerful influence of viscosity due to sediment in suspension. A channel lined with cobblestones will stand a clear-water velocity of 6 to 8 ft. per second, but when it is followed by a more viscous stream heavily charged with material in suspension, although at a reduced velocity of only 3 to 4 ft. per second, the cobblestones will be at once picked up and transported with the flow.

Mr. Hooker. ELON HUNTINGTON HOOKER, Ph. D.—It is to be hoped that advantage may be taken of the opportunity offered by the Niagara River to put at rest the question of surface convexity in a river at a stage of permanency. The excessive velocity in the rapids makes it a favorable place to detect the phenomenon, and at the same time renders it difficult to obtain those cross-sections of surface and bed which would show the convexity, and also the fact that it was not due to local distortions of the bed. The measurements known to the author do not warrant the statement that surface convexity exists at a permanent stage with normal form of bed.

The influence of curves in producing local suspension of sediment has been referred to in the discussion. The principle of scour at the concave bank and deposit at the convexity opposite, occurring in each case below the point of maximum curvature, must have its logical outcome in a longitudinal sinuous movement of the stream bed. M. Clavel* has been able to trace this movement clearly in the Garonne at Caudrot. The sinuousities are marked, and maps of the years 1780, 1806, 1822 and 1840 give opportunities for comparison by superposition, as shown by the plates accompanying his article.

* Annales des Ponts et Chaussées. April, 1895.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 792.

THE RECONSTRUCTION OF GRAND RIVER BRIDGE.

By W. A. ROGERS, Jun. Am. Soc. C. E.

PRESENTED OCTOBER 7TH, 1896.

WITH DISCUSSION.

The reconstruction of the bridge over Grand River, 3 miles south of Chillicothe, Mo., on the Kansas City Division of the Chicago, Milwaukee and St. Paul Railway, by the railway company's Bridge and Building Department, presents several interesting features which will be described by the author, who was the engineer in charge of the work.

The Kansas City Division was built west from Chillicothe during the year 1887. The crossing of the Grand River consisted of a pile approach 1 070 ft. long at the east end, four 138-ft. wooden Howe truss spans across the channel, and a 16-ft. pile approach at the west end. The Howe trusses rested upon pile piers, each consisting of 48 oak piles well braced, planked on the outside, filled with rock and riprapped at the bottom. That these piers were well built is shown by subsequent events. Grand River, at the ordinary stage of water, is

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about 150 ft. wide at the bridge, and from 4 to 6 ft. deep. It is subject, however, to very sudden rises, ranging from 10 ft. to 29 ft. above low water. At times of high water the current is very swift and carries large quantities of driftwood.

In 1894 it became necessary to renew the east pile approach, and it was decided that the Howe trusses must be replaced during the succeeding year. The wooden piers were in fair condition, and, inasmuch as they were good for at least two and possibly three more years, it was also decided that it would be best to replace the Howe trusses with iron spans of the same length, erected on the old piers, and then, when it became necessary to do so, to build new piers as near as practicable to the old ones, and on their completion to transfer the iron spans to them, the plan being to carry the wooden trusses as they were until the end of June, 1895, then to put in falsework and erect the iron during the fall. It was considered unwise to put falsework or strengthening bents under the trusses before this time on account of the danger from driftwood in case of high water. Grand River usually has a high June rise and is expected to stay at a comparatively low stage from then until some time in January.

During March, 1895, the second span from the east began to show evidence of weakness and three strengthening bents were driven under it as a measure of safety. The spring and summer were exceptionally dry, the usual June rise did not occur, and about July 1st it was thought to be safe to begin the falsework preparatory to taking down the spans and erecting the iron in the fall. This was started somewhat earlier than was necessary for iron erection purposes because the timber seemed to be deteriorating faster than had been expected. As a precautionary measure in case of high water, it was to be built last under the third span from the east, that being the channel span. During the middle of August a rise of the river occurred without serious consequences. On August 28th, the falsework was in the following condition. It was complete under the two east spans, and there was a bent of falsework at the third panel point from each end of each of the two west spans. At that date the river began to rise very rapidly, bringing down large quantities of brush, logs and whole trees, and did not stop rising until it had reached a stage about 20 ft. above low-water mark. In spite of all that could be done a jam was started at the bridge, against the falsework, which rapidly increased in size until a

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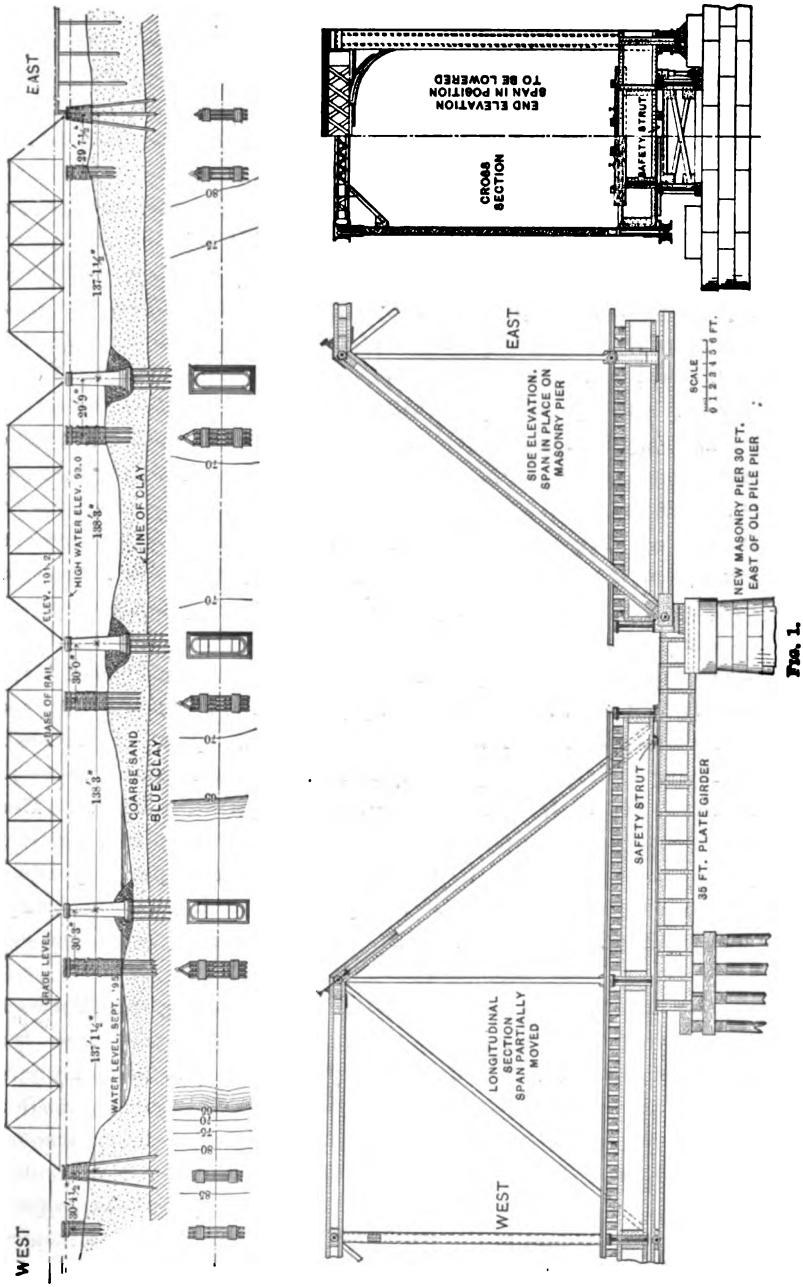

FIG. 1.

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mass of this driftwood from 4 to 12 ft. in thickness, extending the full length of the spans and 700 ft. up the stream, was lodged against the piers and falsework. One of the bents of falsework was carried away and one of the piers was thrown 8 ins., and another 5 ins. out of line at the top. A view of the accumulation of drift is shown in Plate IX, Fig. 1.

On examination it was found that the piers were not undermined by the current, but seemed to have been simply bent out of line by the great pressure exerted against them. They were apparently uninjured by their rough usage during the high water. The river subsided rapidly to its normal depth, leaving a large part of the driftwood on the sand at the sides. It was so intricately interwoven that it was impossible to separate a piece without cutting it loose from the rest.

The lodging of this immense amount of driftwood against the bridge put a new aspect on the situation, and in view of all the circumstances it was decided to begin the erection of the new spans as soon as the iron could be obtained from the bridge works, in order to permit taking out the falsework and clearing the channel, and instead of carrying the iron on the old piers for two or more years, to begin at once the construction of the new ones. It was decided also that the second, third and fourth piers from the east should be built of stone and the first and fifth should be pile piers. The two last are on the banks out of the line of the swiftest current. The piers were located, beginning at the east, respectively 29 ft. 7½ ins., 29 ft. 9 ins., 30 ft., 30 ft. 3 ins., and 30 ft. 4½ ins., east of the old piers (see Fig. 1). This was as close to the latter as it was convenient to work. On their completion the spans were to be moved endwise to their proper position on the new piers. It was with these problems confronting the Bridge and Building Department that the actual work of reconstruction was begun.

The first thing to be done was to finish building the falsework and traveler runway for erecting the iron and to take down the Howe trusses. This was done in September, and in the middle of October the iron crew began building their traveler, but were unable to begin the actual work of erection until November 1st on account of delays in getting the material. These were due in part to the difficulty the bridge works had in obtaining the required shapes from the mills, on account of the number of orders on hand, and in part

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to the scarcity of cars for transporting the iron from the shops to the bridge site. This work was being done at a time when every car was needed to carry on the business of the railroad. Notwithstanding these delays, on November 19th the spans were all erected and riveting finished. The erection of the spans did not present any particularly interesting features. The four spans weighed 714 489 lbs., and the cost of iron and erection combined was 2.73 cents per pound. The cost here includes, as in all other cases where it is given, the amount paid in wages to the railway company's employees, and the cost of the material delivered at the most convenient point on the company's lines, and no other items. Iron and stone were purchased by weight, and timber by board measure, free on board cars on the company's lines.

Many schemes for disposing of the driftwood and clearing the channel were proposed. Several contractors asked permission to bid on the work, and offers ranging from $1 900 to $14 000 were made. It was decided, however, that the railway company was as well equipped for the work as any contractor; that it could do the work, which would be as novel to the contractor as to it, just as cheaply; that there was considerable uncertainty as to the cost; some risk to the bridge in case an attempt to burn the drift should be made, and that, all things considered, it would be unwise to contract for its removal. On November 11th the work of removal was begun in an experimental way, with the understanding that it should in no way be allowed to interfere with the building of the masonry. In preparing the foundations it was necessary to remove considerable drift which had lodged on the site of the new piers. This was done largely by hand and by teams, the wood being hauled to one side, piled and afterward burned. On November 14th a small crew of men and teams started to clear that which was piled against the wooden piers on the east side of the channel. The plan was to pull all of it which had lodged against the piers from them first; next to clear a space alongside and parallel with the bridge, in order to remove any danger in case it was decided to burn the wood, and then to set fire to a part of it to see if it would burn in a body. The method of procedure was to drag the logs with teams through the bridge to the sand bank below, after loosening them by chopping and the use of blocks fastened to the bridge. Fires were kept con-

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tinuously burning the wood thus hauled, so that each night practically all which had been hauled out during the day was consumed. There was considerable fine stuff, twigs, small branches and cornstalks in with the logs, which caused much trouble, as it all had to be picked out by hand and hauled to the fires in wagons. It had to be taken out, however, in order to get at the logs.

On November 21st the driftwood around the piers east of the channel, and also a strip about 30 ft. wide parallel with the bridge, had been disposed of. The wind was blowing strongly from the bridge, and after trying one corner of the driftwood it was decided to set fire to all of it lying east of the channel. About 2 o'clock in the afternoon, after placing a steam-pump and 250 ft. of hose to protect the bridge, fires were started on the side next to the bridge and along the channel in a large number of places. In a few minutes the whole mass was in a blaze. It was a magnificent sight, burning rapidly, but without high flames or sparks. From time to time the wooden piers were wetted down to prevent their catching fire. By 6 o'clock in the afternoon the fine stuff was practically all consumed, and the logs were burning well, and it was in such shape that it was safe to let all but six men and the pump engineer go home for the night. The fire cleared up the driftwood east of the channel in good shape, except that here and there a pile of it was left unburned, and at the upper end quite a large section did not burn at all, due largely to the fact that the wind changed about the time the fire reached it, blowing the flames away. The fire did not cross the river.

The wood left unburned east of the channel was next collected, the idea being that after clearing each side of the channel it would be unnecessary to touch that which was in the water, as in case of high water it was expected the current would cut a new channel in the cleared space provided on the east side. This has been partly proved, as, during a small rise which occurred during the latter part of December, the river started to cut a new channel along the east side of the driftwood left in the water. This is an advantage, as it will stop the undermining of the west bank. The driftwood was cleared from the west pier, and on November 27th fire was set to that on the west shore. It did not burn as well as that on the east side on account of the fact that there had been snow. Some experiments were made with crude oil, but with little success. After disposing of

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the drift on each side of the channel the long logs lying in the channel which were accessible were cut up, and it was considered perfeclty safe to suspend work upon it.

It was believed that inasmuch as this driftwood came down at a stage about 20 ft. above low water, very little more would come down until the same stage was reached again, and that as soon as the river began to rise the remaining drift would begin to float off, and by the time the river was high enough to bring any more, that lodged in the river, now there were no obstructions, would have floated away.

The cost of disposing of the driftwood was very small compared with the lowest estimate, the total cost for labor and material used being $938 78.

It was thought best to wait until the east iron span was erected before beginning the construction of the masonry, in order to avoid the confusion incident to two crews working at one end of the bridge at the same time. The east pier was selected to begin with on account of the necessity of locating the stone yard on the level ground just east of the east span. The stone was unloaded directly from the cars to the stone yard by a hand-power derrick, except for a short time toward the completion of the work when steam power was used. The pile bridge was of such height that the work of the yard derrick in unloading and piling the stone was lowering. Raising stone to any considerable height by a hand-power derrick is very expensive. The stone was delivered from the stone yard to the piers by a push car running on an elevated track made of timbers taken from the trusses and built so as to pass close to the south end of each pier. It was on the ground at the stone yard and was given enough grade so that the car loaded with stone needed only one man to manage it. The stone on the car at the pier was of such height that the work of the setting derrick was all lowering except for a few of the top courses.

The soil at the site of the three masonry piers consists of coarse sand extending to about 10 to 12 ft. below low water and then a stratum of blue clay. The foundations of the east and middle piers consist of 44 pine piles each, from 19 to 23 ft. long, extending from 10 to 16 ft. into the clay. The piles are cut off 2 ft. 2 ins. below low water, capped with 12 x 12-in. pine timber, and these covered with a platform of 8-in. timber. The piles were driven with a steam-power land driver, having a 3 000-lb. hammer, aided by a water-jet. The

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latter was of material assistance in driving the piles through the sand. A 6 x 5½ x 6-in. duplex Worthington pump with a 4-in. suction and a 2½-in. discharge was used to furnish water for the jet.

The foundation of the west or channel pier consisted of 44 pine piles from 21 to 23 ft. in the ground and cut off 4 ft. below low water. On these piles a caisson was sunk made of three thicknesses of 8-in. timber for the floor and 3-in. plank for the sides. Most of these plank had been used in the falsework as sway braces, and most of the piles used in the foundation of the three piers had also previously been used in the falsework. The masonry had courses ranging in thickness from 15 to 26 ins. with beds and joints not exceeding ⅜ in. Western Portland cement was used to set the footings, the lower three neat work courses, the coping and bridge seat stones. The rest of the courses were laid in Milwaukee cement. The mortar was composed of one part of cement to two parts of sand by actual measurement. The sand was obtained at the site and was an ideal mason sand. The stone used is a limestone obtained at Stone City, Ia., and is the stone which this railway company has principally used for its bridge work for some time past. It is soft, easily cut, and very durable. Frost has little effect on the ledges used for the company's work. Displacement of the hip stones when struck by logs or driftwood is prevented by securing each by one or more dowels extending about 8 ins. into the course below.

The foundation work was begun November 5th, 1895, and was completed January 2d, 1896. The stone cutting for the piers was begun November 7th, 1895; setting stone on the east pier was commenced November 24th, and the masonry was finished January 18th, 1896. The total cost of the foundations of the three piers was $2 374 40, or an average cost of $18 per pile driven. There are 715 cu. yds. of masonry, costing $5 747 30, or $8 04 per cubic yard. The cost per cubic yard, including the cost of foundation, was $11 36.

In designing the iron spans, the fact that they were to be moved longitudinally after erection was considered and the plan of moving was decided upon. In furtherance of this plan, the end floor beams were to be attached to the end shoes by means of vertical angles in such a way that the spans could be lifted and carried by them, the plan of moving being to pull the spans, one at a time, on small rollers running on the top of a 35-ft. iron girder span reaching from the old

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to the new pier at each end of the span being moved. For this purpose two 35-ft. plate girder spans were constructed after the general plan of one of the company's types of deck girder spans, except that the weight was increased about 14%, and on the top of each girder a ½-in. plate 10 ins. wide was riveted with countersunk rivets. The girders were spaced 9 ft. center to center. The cross-frames and laterals were to be connected by means of bolts, so that the girders could be placed in position, bolted up, and after use unbolted and taken out separately. These girders are to be put in service as ordinary bridges now that the spans have been moved. Twenty-eight 2½-in. rollers, with shoulders projecting ½ in., were made to run on the top of the girders. These rollers were 10½ ins. between shoulders, and 12 ins. long over all. They were made to fit the plate on the top of the girders closely, so that they would run true while in use. Four shoes 12 ins. long and 10 ins. wide, to be clamped to the lower flange of the end floor beams directly above the moving girders, were made to fit between the shoulders of the rollers. Two struts of two 6 x 4 x ½-in. angles each were clamped to the lower flanges of the stringers a short distance from the end floor beams. When in position, with the span raised and resting on the rollers, each strut extended over the top of the moving girders, clearing the top plate ½ in. both above and at the side. Its province was to catch the span after a drop of ½ in. and to prevent any lateral motion in case the shoes ran off the rollers.

Wooden shores were to be placed between the top of the second floor beam from each end and the top chord to prevent buckling of the span in this case. Fig. 1 shows a span partially moved and gives the details of the moving device. Each end of the span in process of being moved was to be raised by means of four 25-ton hydraulic jacks, two on each side, under the end floor beams, between the moving girders and the end shoes. The rollers were then to be placed under the shoes on the girders and a locomotive attached to the span to pull it ahead. The plan contemplated was to pull the spans one at a time at intervals measured by the length of time necessary to take up the moving girders at the front of the span last moved and to place them in position under the rear of the span next in order. Each moving girder span between the truss spans would carry the rear end of one span and the front end of the next span without being shifted. During the intervals, after moving one span and while preparing to move

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the next one, the track between these two spans was to be carried on blocking resting on the moving girder span in position between them.

The ends of the girders resting on the new piers were placed at such a height that when the truss span had rolled into position it would be 0.5 in. higher than when lowered to the pier. The other ends on the old piers were raised 1 in. higher, in order to give a downgrade to assist in the moving. This was effected by putting shims of the proper thickness under the ends of the girders resting on the new masonry piers and on the wooden piers at the expansion ends and by cutting down to give the proper elevation at the fixed ends. Between February 1st and 8th all work preliminary to moving the spans was completed and everything put in shape, so that when the moving was once started there would be no delays on account of lack of preparation. On the afternoon of February 8th, which was Saturday, the track ties between the east new pier and the east old one were cut off and the stringers shoved in far enough to permit setting the first girder span early in the morning of February 10th. The first thing done on that date was to set in place the girders of this span, using the derrick cars which have been specially designed by the Bridge and Building Department for iron bridge erection. This span was to carry the front end of the east truss span. After landing these girders, those for the rear end of this span were put in place, which was a more difficult task, inasmuch as the girders were 35 ft. long and the truss panel length was only 22 ft. 6 ins. It was accomplished by leading them endwise, by means of the derrick cars, through the end panel between the batter post and the hip vertical.

The two girders at the front end of the span and one of those at the rear end were lowered into position by noon. In the afternoon the other girder was placed, the cross-frames and laterals of both spans put in and the bolting up almost completed. During the next morning this work was finished and the girder spans were centered exactly to position. The center of the end on the old pier was made to coincide with the center of the truss, and the end on the new pier was centered with reference to the center line desired for the new position of the bridge, which was $1\frac{1}{4}$ ins. south of the old center line. This would cause the spans to move laterally $1\frac{1}{2}$ ins. while going 30 ft. longitudinally.

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The piles between the east end of the truss span and the east new pier were cut out and the track blocked up on the moving girder span. The tackle with which the locomotive was to do the pulling was fastened to the end batter posts and laid ready for use. Everything was made in readiness to move this first span between 12.45 P. M., when passenger train was due from the west, and 3.20 P. M., when another passenger train was due from the east. After the passage of the train at 12.45 P. M. the track was torn out ahead of the span and the material piled at the east end of the second span, in the order in which it would be needed to block between the first and second spans. While the carpenter crew was doing this work the iron crew was engaged in jacking up, first the east and then the west, end of the span, clamping the roller shoes on the lower flanges of the end floor beams, then lowering them down on the rollers placed on top of the girders. The safety struts were put in place and the locomotive attached to the steel cable by which the span was to be moved. This cable was attached at one end to the batter posts a short distance above the level of the track. It was so arranged with a double and triple block as to give six parts to the line. This was done to reduce the speed of moving slower than it would be practicable to run the locomotive. The other end was fastened to a chain which was attached in turn to the drawhead of the locomotive by passing a coupling pin through one of the links. In order to stop suddenly if it were wished to do so, block and tackle were fastened between the rear batter posts of the span to be moved and the front batter posts of the next span, and men stationed at these lines for that purpose. The force was placed as follows:

The conductor and brakeman of the work train near the engine to pass the signals to the engineer.

Three men to look after the moving tackle.

Two men at each moving shoe, one of whom fed the rollers in front of the shoe as the other passed them to him as they came out behind; each shoe rested on five rollers, thus leaving two extra ones.

One man stood at each end of the span where he could watch the operation of the rollers at his end and give signals to regulate the speed when necessary.

Two men were detailed to pay out the snub lines at the rear.

One man was sent out in each direction to flag approaching trains.

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The carpenter foreman and crew of eight men were stationed at the front end of the second span to block up on the moving girders and to put in the temporary track as fast as the gap behind the span opened.

At 1.43 P. M. the locomotive started to pull the span and at 2.02 it had reached its proper position ready to be lowered to the new piers. At 2.40 P. M. the spans were let down on the piers, the track connected up at the east end of the span and everything in shape for traffic. At 2.55 P. M. an eastbound freight train crossed the bridge. The actual time during which the track was disconnected was 1 hour and 55 minutes. The time consumed in pulling this first span across the distance from the old to the new piers was 19 minutes. The girders at the front end of this span were picked up and loaded on the cars before night, ready to set down at the rear of the second span the next morning. Those in the rear of the first span were all ready to carry the front of the second span.

The next day the weather was bad and no work was done. On the following morning the girders were lowered into place at the rear of the second span. This span was moved after 3.20 P. M. the same day. The operation of moving the second, third and fourth spans was practically the same as that of moving the first. The last two spans were moved on February 14th and 15th respectively.

TABLE GIVING TIME CONSUMED IN MOVING EACH SPAN.

Span number from east.	Date when moved.	Track was disconnected at.	Started to pull span at.	Span reached new position at.	Track ready for traffic at.	Length of time track was impassable.	Length of time required to pull span from old to new position	Distance moved by span.
		P. M.	P. M.	P. M.	P. M.	H. M.	Min.	Ft. Ins.
No. 1...	February 11th, 1896....	12.45	1.43	2.02	2.40	1 55	19	29 7½
No. 2...	" 13th, "	3.30	4.06	4.16	4.45	1 15	10	29 10
No. 3...	" 14th, "	3.30	4.18	4.24	4.55	1 25	6	30 1
No 4...	" 15th, "	3.30	3.49	4.16	4.50	1 20	27*	30 4½
	Average.....	.....	.....	.....	.....	1 20	15½	

* Fifteen minutes' delay due to a shoe working loose and becoming skewed.

A view of the last span when half moved is shown in Plate IX, Fig. 2.

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On February 17th the girders were picked up and loaded on the cars for shipment.

The time consumed in moving the four spans was 6½ days. The locomotive used was a Rhode Island 17 x 24-in. cylinder, eight-wheel engine, weighing 86 150 lbs., 54 450 lbs. on the drivers. The engineer reported the pull to be very light. There was no trouble in starting or stopping.

The total cost of moving the spans was $716 81, or an average of $179 20 per span. After moving, the track was lined, the floor finished, the old piers removed and their rock filling used to rip-rap the new piers. Everything was finished March 15th, 1896. A statement showing the cost in detail is given at the end of the paper.

During the reconstruction of this bridge there were no delays to passenger trains and rarely to freight trains, and these never exceeded ten minutes. During the erection and moving of the iron spans each train was required to come to a full stop during working hours and receive a signal before crossing the bridge, and to reduce speed while crossing the bridge during the night. The rest of the time trains reduced speed during working hours only.

Notwithstanding the unusual character of part of this work it was all done on week days. Experience in the Bridge and Building Department has shown, aside from questions of principle, that as a rule Sunday work is more expensive and no safer than that done on week days.

Plans and specifications, including all working drawings, were made by the Bridge and Building Department of the Chicago, Milwaukee and St. Paul Railway Company. All work at the bridge site was performed by the company's employees, and every item of construction from beginning to end was carried out by the department.

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CONDENSED STATEMENT OF COST.

Falsework, taking down Howe Trusses and Repairing old Piers.

Labor	$1 834 99		
Train service	150 00		
		$1 984 99	
Material		1 606 90	
			$3 591 89

Two Pile Piers.

Labor	$287 00		
Train service	40 00		
		$327 00	
Material		420 24	
			747 24

Three Masonry Piers.

Foundation—			
Labor	$1 773 00		
Material	601 40		
		$2 374 40	
Stonework—			
Labor	$3 485 53		
Material—			
282 sacks Western Port. cement	$109 65		
269 sacks Milwaukee cement	67 25		
34 306 cwt. bridge stone	2 061 93		
Miscellaneous	22 94		
	2 261 77		
		5 747 30	
			8 121 70

Ironwork.

Labor erecting	$1 952 97		
Train service	96 78		
		$2 049 75	
Material—			
700 009 lbs. wrought iron	$1 7216 91		
14 489 lbs. cast iron	195 43		
Miscellaneous	21 06		
		17 433 40	
			19 483 15

Floor.

Labor		$292 82	
Material		1 051 32	
			1 344 14

Moving the Spans.

Labor	$521 99		
Train service	58 20		
		$580 19	
Material—			
2 523 lbs. iron (shoes, struts, etc.)	$69 38		
2 445 lbs. iron (increased weight of two moving girder spans)	67 24		
		136 62	
			716 81

Removing old piers, pulling piles, taking out falsework	593 05
Removing driftwood	938 78
Night watchman	465 50
Engineering, office expenses and inspection of stone and iron	1 145 96
Total cost	$37 148 22

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Analysis of Cost.

The falsework, taking down the Howe trusses and repairing the old piers cost $6 53 per lineal foot.

The new pile piers cost $373 62 each.

The foundation of the three masonry piers cost an average of $18 per pile driven.

The stonework cost $8 04 per cubic yard, exclusive of foundation, and $11 36 including foundation. Each cubic yard of finished masonry required 48 cwt. of stone in the rough, and the cost of the labor per cubic yard averaged $4 87.

The erection of the iron spans cost 0.29 cent per pound, and the erection and material together cost 2.73 cents per pound.

The floor of the iron spans, including the heavy iron angle guard rail, cost in place $22 22 per thousand feet, board measure, of timber used, or $2 44 per lineal foot of floor.

The cost of moving the four spans averaged $179 20 each or 0.1 cent per pound.

The engineering, office expenses and inspection of stone and iron formed 3% of the total cost.

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CORRESPONDENCE.

Mr. Bates. ONWARD BATES, M. Am. Soc. C. E.—The author has wisely omitted the details of design, the knowledge of which may be acquired in office practice or in the text books, and confined himself to the practical execution of the plans. The elements of time consumed, labor and material employed and costs, all have to be taken into consideration in the execution of similar work, and information of this class contained in the paper is especially valuable to those who are in charge of such work. Good judgment seems to have been exercised in laying out the work, and this is in a measure checked by the reasonableness of the items of cost. The care with which the work was planned and executed is shown by the use of safety struts, which would permit of a vertical drop of only ¼ in. and no lateral motion in the event of the spans running off the rollers, and in the nice adjustment of grade in the plate girders which acted as a track, reducing the friction in moving the spans.

The problem of the removal of such a mass of driftwood interlocked and settled in the sand, as described in the paper, appeared a difficult one when $14 000 was asked by a contractor for its removal, and the author was favored by the condition of low water which permitted the drift to be burned. If the stage of water had been high enough to keep the driftwood wet, it is probable that $14 000 would have been none too much as the price for its removal. The writer, who has some personal knowledge of this structure, is able to say that during the high water of June, 1896, all of the driftwood which would float away was lifted and carried off. Some of it, however, had become so imbedded in the sand that it remains there and had the effect of turning the main channel away from the west bank and carrying it under the second span from the east end, which is the most desirable location for it.

Mr. Low. EMILE LOW, M. Am. Soc. C. E.—The extremely short life of the original Howe truss bridge built across the Grand River is notable. The author states that these spans were erected in 1887, and replaced by iron in 1895, being only eight years in service. Referring to Plate IX, Fig. 1, which shows an end view of the original structure, it is apparent that the bridge was not housed in. Exposure to the weather evidently hastened the decay of the timbers, the life of which would undoubtedly have been prolonged had precautionary measures of some kind been resorted to.

The author does not mention the kind of timber employed in the trusses, but it was probably some species of southern pine, which, it seems, cannot be relied upon in exposed situations for a longer period than about eight years. In this respect it does not compare favorably

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with the pines of the Northern States, as the writer has knowledge of bridges lasting more than double this period. Timber structures on a southern railroad with which the writer was connected, consisting of ordinary trestles and Howe truss bridges, were erected during the years 1887 to 1890. The Howe trusses were uniformly constructed of southern pine, while the trestles were almost exclusively built of white oak, with southern pine stringers, except in a few trestles where oak stringers were used. These pine stringers are now showing evidences of decay, and in consequence numbers of them are being taken out and replaced. Many sills have also been taken out and new ones substituted. These sills rested upon blocks 4 ins. thick, supported upon stone masonry, the tops of which were well above the surface of the ground. The original sills were in one piece, 10 x 12 ins. in cross-section, while the new ones are composed of two pieces, each 5 x 12 ins. in section. In addition to the stringers and sills, large numbers of the cross-ties in these trestles had to be removed. Mr. Low.

The parts which need to be replaced are those lying in a horizontal position. The upright parts, such as posts, are still serving their purpose, and are not showing very perceptible signs of failure, the only exception being that some of the oak posts were so badly warped as to necessitate their removal. The length of these pieces is about 26 ft., and they are from 10 x 12 ins. to 12 x 12 ins. in section.

From an examination of the Howe trusses the fact is developed, that while portions of the chords are showing signs of decay, the braces are comparatively sound.

The importance of suitably covering wooden bridges does not seem to be well recognized. It cannot be denied that this protection adds materially to the length of their usefulness. Timbers which, from their position in the structure, are more susceptible to decay, should be treated with preservatives of some kind. Owing to the peculiar construction of wooden structures it is not always easy or economical to replace a defective piece, such repairs oftentimes necessitating as elaborate and expensive scaffolding as would suffice for the substitution of a large number of pieces.

Another great advantage gained by covering wooden structures, and especially the stringers, is the immunity afforded from fire. In an instance which came under the writer's observation, a stringer of a certain trestle caught fire from a passing train, but was fortunately discovered before any great harm had been done. In making repairs, instead of taking out the injured member, it was left in position, and additional new pieces put in, one on each side, leaving the charred piece in the middle. On account of its condition it was extremely liable to ignition, and it was not long afterwards that the same trestle was again on fire. This, however, was not discovered in time and resulted in the wrecking of a freight train, which, being loaded

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Mr. Low. with goods of an extremely valuable character, caused a considerable loss.

Referring to the analysis of cost at the end of the paper, the great decline in the cost of bridges since 1887 is forcibly illustrated. The writer desires to compare the cost of Howe truss bridges erected in that year with the cost of iron spans now at the figures given by the author. He recalls one Howe truss span of 76 ft. which cost to erect in 1887 $27 75 per linear foot, or a total of $2 109. Assuming an iron bridge of the same span to weigh 60 000 lbs., at the price given by the author, 2.73 cents, the cost would amount to $1 638, not including the floor.

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FIG. 1.

FIG. 2.

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No. 793.

SUSPENSION BRIDGES—A STUDY.

By GEORGE S. MORISON, Past-President Am. Soc. C. E.

PRESENTED OCTOBER 21ST, 1896.

WITH DISCUSSION.

Fifty years ago the suspension bridge was regarded as the one class of structure adapted to spans of unusual length. Highway suspension bridges were built in almost all parts of the world; several railroad suspension bridges were proposed, and the one actually built across the Niagara River has done service for nearly forty years. Fifty years ago metallic bridge construction was in its infancy, and anything beyond the limit which could well be built of wood was considered an exceptional span. Athough there were a few striking exceptions, 200 feet was practically the limit of wooden truss bridge spans.

The introduction of iron bridges changed these conditions, and a 400-ft. iron span was as readily built as a 200-ft. Howe truss. The

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first 400-ft. span in America was constructed by Albert Fink, Past-President Am. Soc. C. E., in the bridge across the Ohio River at Louisville, Ky., where it is still in use. The cheapening of the price of iron, the increased capacity of rolling mills, and the new methods of making steel, have rendered it an easier task to build a truss of 600 ft. span now than it was to build a 400-ft. span then. The result of this development has been that trusses have superseded suspension bridges, and where a suspension bridge would have been built forty years ago a steel truss is built now.

Furthermore, the old suspension bridges were highway bridges, and highway traffic does not enter upon a bridge as a concentrated load, while it is generally light in proportion to the dead weight of the bridge. Important modern bridges are generally railroad bridges, and the suspension bridge has not been considered stiff enough to serve this purpose, this want of stiffness being largely due to the fact that on a single-track railroad bridge the whole moving load comes on as a concentrated load, and that on railroad bridges of moderate span the moving load is large in proportion to the dead load.

No important suspension bridge has been built since the East River Bridge, the design of which was shaped by the elder Roebling before his death in 1869. During this time great advances have been made in all other forms of bridge construction. While it is admitted that there is no field for suspension bridges of the dimensions of which they were formerly built, a new field is opening in structures of enormous size, bridges that would bear the same relation to a 600-ft. steel truss that the 1 000-ft. suspension span at Cincinnati bore to the 200-ft. truss spans built about the same time in the same neighborhood across the Great Miami.

If, at the present day, an 800-ft. span is to be built, a truss on the beam or cantilever principle would be used, but a 2 000 or 3 000-ft. span would be a case for a suspension bridge. Two things must be remembered. In a suspension bridge of any such enormous size the dead weight would be large in proportion to the live load, and the distortion due to the passage of trains would be comparatively small. Such a bridge would be built for two or more railroad tracks, its length would be much greater than that of a single train, and the conditions under which a concentrated maximum load would advance as a whole, upon the bridge would be rare. In the manner of the passage of loads

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such a bridge would resemble a highway bridge more than a short-span railroad bridge.

Before, however, a suspension bridge is built which will bear the same relation to the modern steel truss that the old suspension bridge did to the wooden truss, the same advance in details must be made in suspension bridges that has been made in truss bridges. If such a bridge is to be built now, the designer must concentrate in the work of a single design all the improvements corresponding to those which truss bridge builders have spent many years in developing.

This paper is submitted with a view to opening the way for improvement and to show that a great suspension bridge, which would be well adapted to railroad service, would involve no insurmountable difficulties of construction.

The method of demonstration and illustration which has been adopted is the explanation of the design of a suspension bridge of unusual dimensions and capacity. The size selected for this design would give a clear opening of about 3 000 ft., this corresponding to the dimensions proposed for the North River at New York. The plan discussed is simply a general plan, but as such a discussion to be valuable must be accompanied by estimates, the depth to rock at the sites of the towers has been assumed to be 140 ft. below mean high water; this corresponds to the depths which borings have found opposite the foot of Seventieth Street. It has been assumed also that the anchorages would be built on rock, and elevations have been assumed for these anchorages. These elevations correspond with reasonable accuracy to the position of the bluffs on the west side of the river opposite Seventieth Street, and are probably not very different from the position of the rock on the east side at the same location, but it is not likely that a structure located there would be of the symmetrical character which is now described. The design gives a clear headroom of 150 ft. at all stages of water.

While these conditions correspond more closely to the location at the foot of Seventieth Street than to any other place, this paper is intended as a study of suspension bridges and not as an approval of any particular location. The location at Seventieth Street undoubtedly has great advantages, especially in the matter of anchorages. A location below Thirty-fourth Street has also very great advantages, particularly in the matter of convenience to existing business centers.

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It must be observed that the distinctive feature of a suspension bridge lies in the fact that it has no compression member. The weight is carried by cables stretched from tower to tower, which are secured by heavy anchorages at each end. The strains in the cables tend to overturn the anchorages, and any motion of the anchorages disturbs the entire structure; a slight settlement in the towers might not do much harm; any motion in the anchorages is felt through the entire bridge. In this respect the conditions of a suspension bridge are the opposite of those of a cantilever bridge. Any settlement in the towers of a cantilever bridge is exaggerated at the ends of the cantilevers, while the anchorages are usually of such character that they can easily be adjusted. It is therefore of the utmost importance that the anchorages of a suspension bridge should have foundations which will not yield and which can be put in at reasonable cost. These conditions are assumed in the estimates in this paper. A study of the paper, however, will show that the form of anchorage proposed admits of great latitude in use, as the inclination of the backstays becomes independent of the inclination of the main cables, and the anchorage for one pair of cables may be placed much farther back from the tower than the other pair. This might present great advantages if it became necessary to locate anchorages in a portion of the city already built up.

The capacity of the bridge designed has been reached in a backhanded manner. The bridge has been designed to carry a total load of 25 tons or 50 000 lbs. per lineal foot. The design has then been developed and the dead weight calculated, and the result is a balance for the live load of 11 000 lbs. per foot over the entire structure. As the width of the structure is 92 ft. between the stiffening trusses, this corresponds to about 120 lbs. per square foot of floor. If this space were to be occupied by eight railroad tracks it would amount to 1 375 lbs. per lineal foot per track, which exceeds the weight of any passenger train. It would amount in the aggregate on a length of 3 100 ft. to 34 110 000 lbs., equivalent to eight freight trains 1 400 ft. long, each weighing 3 000 lbs. per lineal foot. It is probable that the requirements of any location where a bridge of this magnitude would be considered, would be satisfied by four railroad tracks adapted to a heavy class of traffic and four rapid transit tracks to be operated by electric cars or short trains of a character which would require only a floor stiffener to secure the necessary rigidity. Therefore, in propor-

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tioning the stiffening truss the variable load has been taken on the basis of 12 000 lbs. per lineal foot, corresponding to a load 3 000 lbs. per foot on each of the railroad tracks, with no provision for unequal weight on the rapid transit tracks, or to 1 500 lbs. per lineal foot on all eight of the tracks. These provisions correspond to four maximum freight trains or eight maximum passenger trains.

In one respect the design departs radically from suspension bridges hitherto built. The cables, instead of being made of straight wires, are made of ropes, and these ropes, instead of being passed over the towers and around pins in the anchorages, are socketed, both at the top of the towers and in the anchorages, all connections being made through the sockets. This modification is really the essential feature of the whole design. The objections which will be raised to it are, first, that a straight wire is both stronger and less extensible than a twisted rope made of the same wire; and second, that no socket can be made which will develop the full strength of the rope. Both of these objections are true, but a rope can be laid in such a way that the modulus of elasticity is only about 1 000 000 lbs. less than that of a straight wire, and a rope can be socketed in a way which can be absolutely depended upon to a fixed amount of strain, and the strength of the structure will then be determined, not by the strength of the wire, but the strength of the connection at the end of each rope. Furthermore, experiments have shown that ropes constructed in the manner proposed have an extremely uniform modulus of elasticity, which is the most important thing. The advantages of this system of construction are principally two; the ropes can be made in the shop, adjusted to length there, carried to the bridge site and put up in the least possible time; the wires are practically straight from one end to the other, the decided turns required over saddles and the short turns required around pins being entirely avoided. With this arrangement the objections to a strong stiff wire are removed.

Another feature which is believed to be novel is the method of holding down the ends of the stiffening truss. When one-half the span is loaded, the upward reaction at the unloaded end is equal to the downward reaction at the loaded end, so that the stiffening truss must not only be supported but anchored down. The stiffening truss of this design is made 1 000 ft. longer than the span, thus extending 500 ft. back toward the shore from each tower, while the suspenders in the

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150 ft. next to each tower are omitted. The result is that the duties of the stiffening truss proper are confined to a length of 2 800 ft.; back of each tower is a span of 500 ft., from which a cantilever 150 ft. long projects to each end of the stiffening truss proper. The reactions of the stiffening truss are taken by the ends of the cantilevers, and the cantilevers are themselves anchored by the weight of the shore spans. This arrangement has the further advantage of leaving 150 ft. between the towers and end suspenders, within which the cables will adapt themselves to any changes of length and height due to temperature, loads or otherwise.

The description of the design follows the order of computation, the cables being first proportioned to carry the selected weight (50 000 lbs. per foot), the towers being proportioned to carry the cables, the foundations being proportioned to carry the towers, the anchorages designed to resist the pull of the cables, and the suspended superstructure to distribute the moving load.

General Design.

The general design is that of a stiffened suspension bridge, the cables to be of wire, the towers of steel on masonry foundations, the structure being stiffened by steel trusses suspended from the cables.

The cables are four in number, two on each side, the length of span between the theoretical intersection points on the top of the towers being 3 200 ft. and the versed sine 400 ft. To secure lateral stability, the two stiffening trusses are placed 100 ft. between centers horizontally, this affording an opening 92 ft. wide in the clear. At the middle of the span the two cables on each side are brought as close together as possible, or 4 ft. between centers, the width at the center of the span between points midway between the two cable centers being 115 ft. At the top of the towers the cables are spread apart to a distance of 28 ft., the width between the centers of the towers being 200 ft. Each inside cable has, therefore, a cradling of 30.5 ft., and each outer cable a cradling of 54.5 ft., the average cradling between 42.5 ft. The backstays are carried from the towers to the anchorages in planes which are tangent to the horizontal projection of the cradled cables, thus splaying the backstays apart between towers and anchorage. By this arrangement the towers are relieved of all transverse strain, and become, as it were, simply gin poles to

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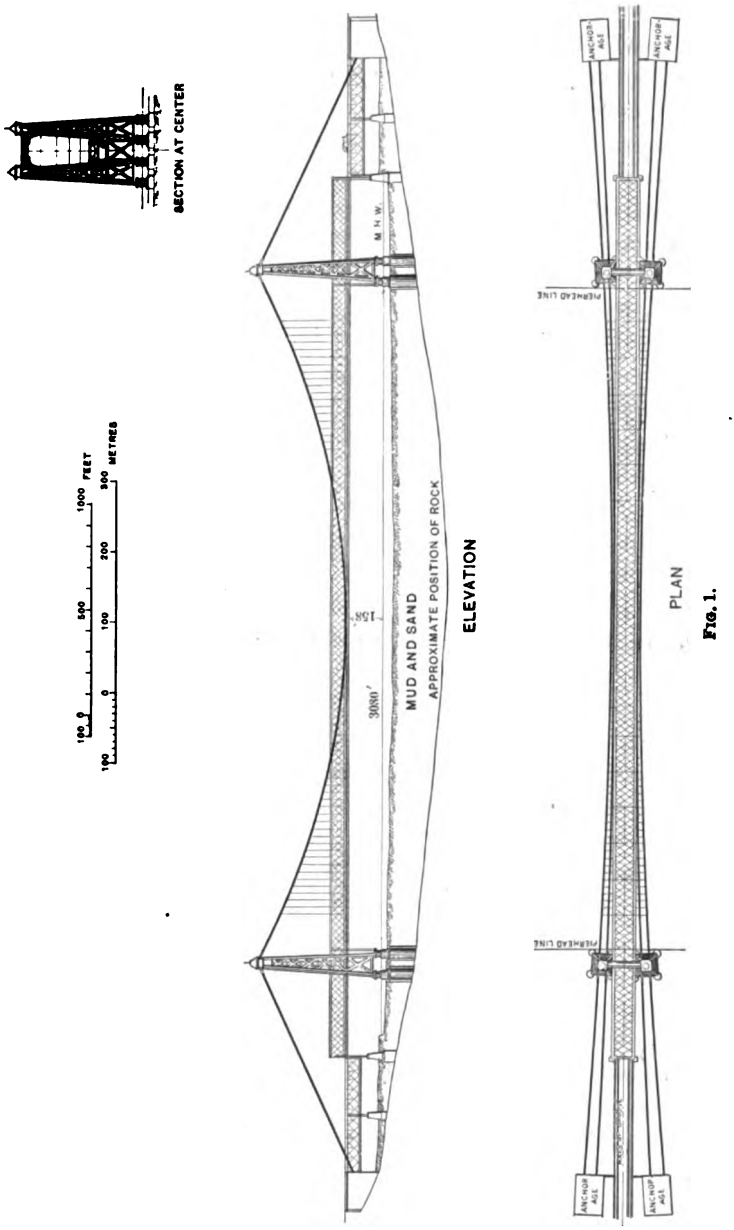

Fig. 1.

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sustain the cables. The lateral stability produced by this arrangement is evident from the plan, Fig. 1.

The towers are of steel, each really consisting of two independent towers formed of four posts, 94 ft. square at the base and battered together so as to be 28 ft. square on the top, the two square towers being connected by a cross truss at the top and resting on masonry cylinders at the bottom. The exact shape and location of the half towers are determined by the direction of the cables, the sides of the two half towers not being parallel and the squares being only approximate.

It has been considered important to reduce the number of cables to four, two on each side. If the cables of the main span and the backstays are counted as separate cables, which the detail hereafter described shows them to be, there are four cables, two leading in each direction, terminating at the top of each tower, the number of cables corresponding to the number of posts, so that the weight from each cable is transferred directly to one of the four posts. This requires cables of much larger dimensions than have ever been used, but there is nothing impracticable in making cables of the required size.

The stiffening truss is 4 100 ft. long over all, divided into panels of 33 ft. 4 ins. each, supported for the central 2 800 ft. by suspenders leading from the cables, while the ends are supported on piers 4 100 ft. apart and intermediate supports are taken on rocking bents 3 100 ft. apart. The truss is continuous for its whole length of 4 100 ft., fastened to the cables at the center and free to move longitudinally at each end. It is considered of great importance to use a continuous truss, thus avoiding the difficulties and lost motion of a central hinge. The difficulties of fastening down a stiffening truss are overcome by the end supports, the end of each truss being a 500-ft. span resting on two supports, from which a 150-ft. cantilever projects towards the point where the suspenders begin. The suspended stiffening truss is only 2 800 ft. long and exerts an upward or downward reaction at the end of the 150-ft. cantilever, according to the position of the moving load; the cantilevers are anchored by the weights of the end spans. The stiffening truss is 66 ft. 8 ins. deep between the centers of gravity of the chords, this depth being adopted for reasons given hereafter; it at once secures the necessary rigidity and permits sufficient flexibility to allow a considerable por-

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tion of the irregular moving load to be taken care of by the change of shape in the cables.

The general elevation, Fig. 1, shows a clearance of 158 ft. above mean high water at the center of the span at a mean temperature of 60° Fahr., when the bridge is unloaded. The extreme effects of temperature are to lower or raise the center 3.15 ft., so that the maximum clearance of the unloaded bridge at a temperature of 0° Fahr. would be 161.15 ft., and the minimum clearance at a temperature of 120° Fahr. would be 154.85 ft. A moving load of 11 000 lbs. per lineal foot over the whole span, being the amount hereafter calculated on, and using a modulus of elasticity of 27 000 000 in the cables, will cause a deflection of 3.96 ft. at the center, thus reducing the minimum possible clearance to 150.89 ft. This load is, however, excessive, and the maximum load which should be estimated in calculating clearances is 9 000 lbs. per lineal foot, which will cause a deflection of 3.24 ft., making the minimum clearance 151.61 ft. The bridge is designed with a camber of 6 ft. in the center of the span when unloaded at mean temperature; this corresponds to a camber of 9.15 ft. when unloaded at the lowest temperature and to a camber of -0.39 ft. when fully loaded at the highest temberature. The maximum camber curve corresponds to a grade of 1.18% at the ends; but this is so short that it will not affect the passage of trains.

In this design the essential difference between a great suspension bridge and a truss bridge of ordinary dimensions has been borne in mind. The truss bridge of ordinary dimensions is so nearly a rigid structure that the changes which take place in its form under passing loads have little influence on the strains in the several members, and such a bridge can be proportioned on the basis of a rigid geometrical skeleton. A long span suspension bridge necessarily changes its shape with every change of load, and changes too in such manner as to relieve local strains, every unstiffened suspension bridge having some shape of perfect equilibrium for every possible loading. These changes of shape play an important part in proportioning a suspension bridge, and so long as they are kept within limits which do not disturb convenience of operation, they are a source of strength instead of weakness. A suspension bridge must be permitted to change its shape within proper elastic limits, and this change of shape must be made the basis of calculations in proportioning the structure.

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Capacity.

The bridge has been proportioned to carry a total load of 50 000 lbs. per lineal foot, which is equivalent to a stress of 40 000 000 lbs. on each of the four cables at the center of the span. The actual dead weight of the cables and suspended superstructure is about 39 000 lbs. per lineal foot, thus leaving 11 000 lbs. for moving load.

The width in the clear between trusses is 92 ft., which will provide for two double-track railroads, each occupying 26 ft., with a space 40 ft. wide between. This 40 ft. can be occupied in various ways; its width is the same as the width between the curbs of Broadway at Twenty-sixth Street; it could be used for four rapid-transit tracks either for street cars or for rolling stock of the same dimensions as that used on the elevated railroads; it could be used as a street with two sidewalks and a roadway between wide enough for four carriages to pass; it could be used for two standard gauge railroad tracks, with a broad promenade for foot passengers between, or it could be used as a driveway with a street railroad track on each side. Another possible arrangement is the construction of eight parallel railroad tracks 11 ft. between centers which is admissible on perfectly straight lines. It is not important to decide how this bridge would be used; enough has been said to show the capacity which would be afforded, and the weights for which it should be designed.

Cables.

The cables are the fundamental feature of the design, and will therefore be described first. The design of the cables departs radically from the features hitherto followed in suspension bridges, and provides a method of constructing suspension bridge cables, under which it is possible to do nearly all the work in the shops, and to diminish field work to a minimum.

The bridge is designed with a versed sine of 400 ft. under maximum load, this being equal to one-eighth the span. On the basis of a uniform load of 50 000 lbs. per lineal foot, or 12 500 lbs. per cable, the stress in each cable becomes 40 000 000 lbs. at the center of the span, and 40 000 000 lbs. multiplied by $\frac{\sqrt{5}}{2}$, or 44 721 360 lbs. at the ends of the span, while the vertical reaction at each end is 20 000 000 lbs.

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Each cable is composed of 253 ropes of equal size arranged in the form of a hexagon with three ropes omitted from each corner; the maximum stress on each rope will therefore be 176 764 lbs. In the design, the ropes are made 2½ ins. diameter, each rope being assumed to have a section equivalent to 3 sq. ins. of solid metal and to weigh 10 lbs. per lineal foot. The stress per square inch on these ropes, will, therefore, be 58 921 lbs., of which $\frac{13}{18}$, or 45 958 lbs., will be caused by dead load, and $\frac{5}{18}$, or 12 963 lbs., by moving load.

Each rope will be a specially laid rope formed of a single straight wire at the center, around which are grouped successive layers of helicoidal wires, so inclined that all will be of the same length, the alternate layers being inclined in opposite directions. When put under strain all wires are equally strained, except the single central wire which acts as a core. This rope bears no resemblance to the ordinary twisted rope. If not larger than No. 8, the wires of each rope can be made continuous from end to end without splicing.

A number of sample ropes were specially prepared by the Trenton Iron Company under the direction of E. G. Spilsbury, M. Am. Soc. C. E., and these ropes were tested at the Watertown Arsenal. The results of these tests will be found in Appendix A. While these tests were of a preliminary character, and the ropes differed in some respects from those which would actually be used in a bridge, they established two important facts: First, that a laid or twisted rope could be made which will develop in the actual rope a strength of more than 180 000 lbs. per square inch of wire, and this with a wire which can be furnished at a reasonable cost; second, that a laid or twisted wire rope of this kind can be depended on in a structure to act uniformly and with a regular modulus of elasticity, the action of the different ropes tested in the latter respect being entirely satisfactory. Twelve ropes were tested in all, of which four were of straight round wires, four twisted round wires, and four twisted wires of the special forms used in a locked rope. The experiments showed the decided superiority of the rope of twisted round wires in all respects except one. The wires were evidently more uniformly strained in this rope than in the straight wire rope, and gave decidedly better results than the peculiar-shaped wires in the locked rope. The only respect in which the twisted rope was inferior to the straight rope was in the modulus of elasticity, which was about 25 000 000 in the

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twisted rope, and 28 000 000 in the straight wire rope. The twisted rope was made with a machine already in existence, and with the twist commonly used on similar ropes. For the special use considered, it is probable that the twist could be reduced to one-half or perhaps to one-third that laid by this machine, and that the modulus of elasticity could be raised to about 27 000 000. The modulus of elasticity does not affect the strength of the structure; the only effect of a low modulus is slightly to increase the deflection. In none of the twelve tests, involving twenty-four sockets, did a rope pull out of a socket, but in nearly every instance the fractures occurred in the outer layer of wires and inside the sockets. An examination of the sockets showed a rough shoulder, which undoubtedly had something to do with this fracture. By a modification of the interior shape of the socket, it is probable that this difficulty could be largely removed, and the strength of the ropes increased from 5 to 10 per cent.

Each rope will be fitted into a steel socket at each end 12 ins. long, the diameter of the socket to be twice the diameter of the rope. By adjusting the ropes under strain at the works, and arranging a special machine to trim the under edge of the socket after the rope is fastened into it, it is believed that the length can be so accurately fixed that no further adjustment will be required in the field. If, however, this cannot be done, the arrangement designed permits the employment of fillers under the square shoulders of the sockets, so that the ropes can be adjusted in position.

There will be four cables in the main span of the bridge. There will be four cables in the backstays on each side of the river. There will, therefore, be twelve cables in all, each of which must be fastened at each end.

The method of fastening the cables is shown in Fig. 2. Fifty feet from each end the several ropes, which are compressed compactly together in the body of the cable, begin to separate so that they are 4.9 ins. between centers at the ends, and the successive vertical sets of ropes are 4¼ ins. between centers. On the top of each tower post is placed a steel casting through which all vertical strains are transmitted, and on this casting rests a 20-in. steel pin. On this pin are set up 20 steel plates 2 ins. thick, each plate measuring 10 ft. in the direction of the axis of the cable and weighing 9 255 lbs. The several

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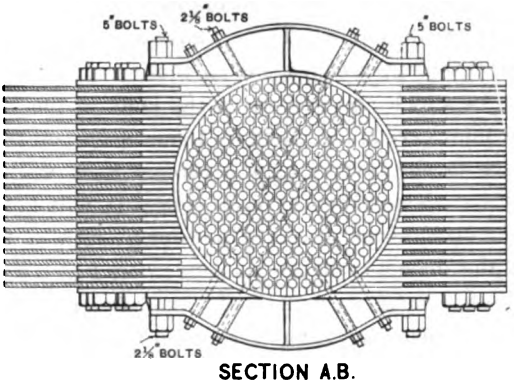

SECTION A.B.

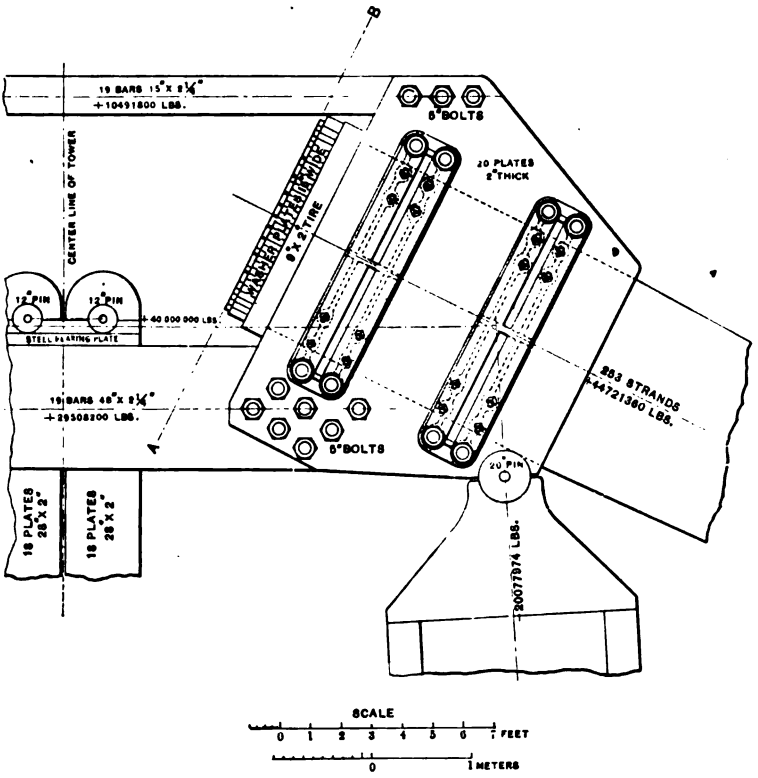

FIG. 2.

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ropes of which the main cable is composed, when spread, pass between these several plates, being held in exact position by cheap cast-iron fillers between the ropes. These plates are machined to a true plane surface on the upper edges, and on these are placed a series of washer plates on which the sockets at the ends of the cables bear. These washer plates are 2¼ ins. thick by 16 ins. deep, and the divisions come in line with the centers of the ropes. Each washer plate is bored out for its whole depth on each side with half holes slightly larger than the diameter of the ropes, and for a depth of 10 ins. with half holes of the diameter of the sockets. (See Fig. 3.) Each rope therefore passes through a round hole, one-half of which is bored in each adjacent washer plate and the socket fits into an enlargement of this round hole, bearing on the annular surfaces between the large

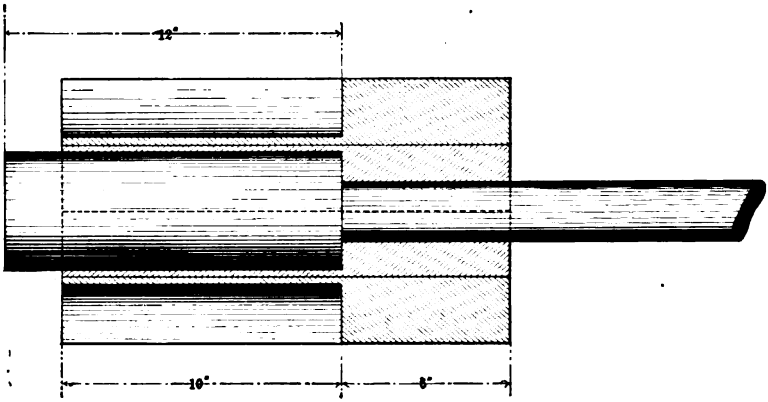

FIG. 3.

and small cylinders. The series of washer plates are bound together by a steel tire shrunk around them. The large plates are bolted together with eight 5-in. bolts and sixteen smaller bolts inclined so as to pass between the ropes, all of these bolts screwing up against heavy cast-steel washers on the outside, the plates being kept at proper distances by the cast-iron fillers.

The entire strain in the cables is transmitted to the large steel plates through the washer plates which bear against them. In the large plates this strain is decomposed into a nearly vertical strain which passes through the 20-in. pin and the steel casting into the post, and a horizontal strain which is taken across the top of the tower to the corresponding backstay connection. For convenience of construc-

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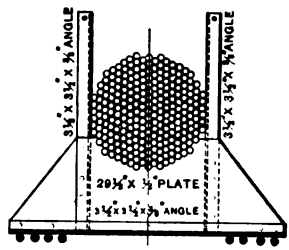
tion and erection this horizontal strain is divided between two tension members, the lower one consisting of nineteen bars each 48 x 2¼ ins., and the upper of the same number of bars each 15 x 2¼ ins., the strain being transmitted to the former by nine 5-in. pins, and to the latter by three 5-in. pins. The full details of this arrangement appear in Fig. 2.

To erect the cables, carrier ropes will be placed above, on which the permanent ropes will be hauled out into place. Eight auxiliary ropes, the general position of which is shown in Fig. 4, will then be run through the unoccupied spaces outside of the washer plates, and on these at suitable intervals a number of iron horses erected. These horses will serve to confine the cable. They are arranged in pairs and braced together, so that they will be stable and will afford room for men to stand and watch the laying of the ropes.

The side washers, large plates and the nineteen lower ties will be put in position, but the spaces between the plates will be open above. The first washer plate will then be put in place and the first rope unreeled and hauled across the river on the carrier ropes. As soon as it is hauled across, the sockets will be dropped into their places in the washer plates. The first three ropes having been laid in this manner, the next washer plate will be put on as well as the necessary fillers; four other ropes will then be run in the same manner and the third washer plate put on. Five ropes will then be run and the fourth washer plate laid. This process will be continued until all the ropes have been laid and all the washer plates are in position, when the washer plates will be finally consolidated by shrinking on the steel tires. The insertion of the diagonal bolts can begin whenever all the ropes below any one of these bolts are laid. The upper 5-in. bolts will be put in when the laying of the ropes is completed. When this is done the nineteen upper tie bars will be placed, bolted up, and the cables are made. By the use of the horses every rope can be put in its final position as fast as laid, and when each layer of ropes is completed men can be sent over them to make sure that they are properly laid, to paint them and prepare everything in readiness for the next layer.



HORSE ON TEMPORARY CABLES DURING ERECTION

FIG. 4.

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The work is condensed as it goes on, and as soon as the last rope is laid each cable is practically complete. Various special appliances must, of course, be worked out. It may even be necessary to use fillers under the socket bearings and adjust each rope by itself. This description, however, shows the main features of the work. It is believed that at least three ropes can be laid daily, and that a cable can be completed in three working months where everything is organized and ready.

As the small tie bars at the top cannot be put in until the cable is completed, it is necessary to hold the broad lower bars so as to overcome the bending strain due to the center of these bars being below the center of horizontal strain. This is accomplished by the use of 36 bars, 28 x 2 ins. in size, which pass between the 19 bars and anchor those bars down to deep cross girders which connect the separate tower posts, as shown in Fig. 5. These vertical ties serve during erection to take out the bending strain in the lower horizontal tie, and, after erection, to bind the whole arrangement rigidly to the top of the tower.

The backstays are of the same dimensions as the main cables and connected at the top of the towers in precisely the same way, the plates to which the backstays are attached being tied to those to which the main cables are attached. In order to keep the cradled cables of precisely the same length, the outer bearings on top of the tower are lower than the inner bearings, as appears in Fig. 6.

Though the backstays are of the same dimensions as the main cables they carry no weight but themselves and run in approximately straight lines (being deflected from absolutely straight lines by their own weight) to and through the anchorages, each anchorage having two tunnels in it through which the backstays run.

At the lower end the ropes of the backstays are spread between plates in the same manner as at the top of the tower, though the details are different because of the direction in which the strains must be transferred. These details are shown in Fig. 7. The strain in the cable is divided into two equal strains, on lines making equal angles with the axis of the cable, and transferred through 26-in. pins to steel castings which bear on closely cut granite masonry, which is built into the coarser material of the anchorage. The lower pin and castings are put in position before placing the cables, but the upper

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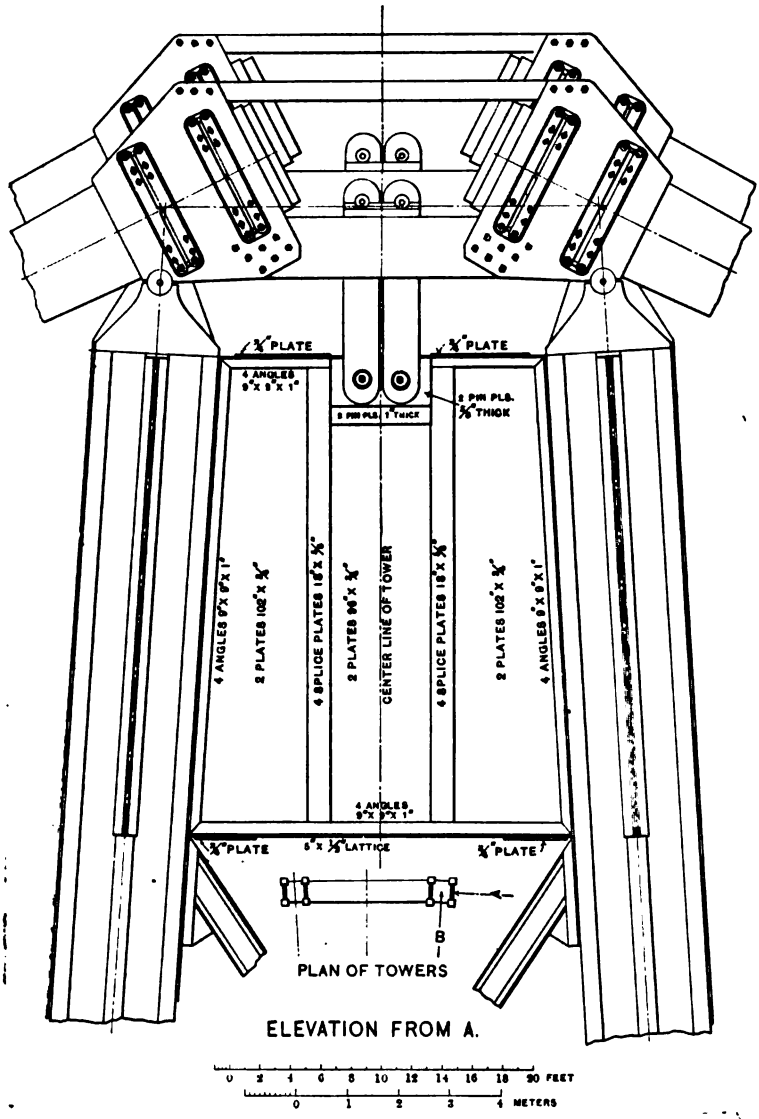

FIG. 5.

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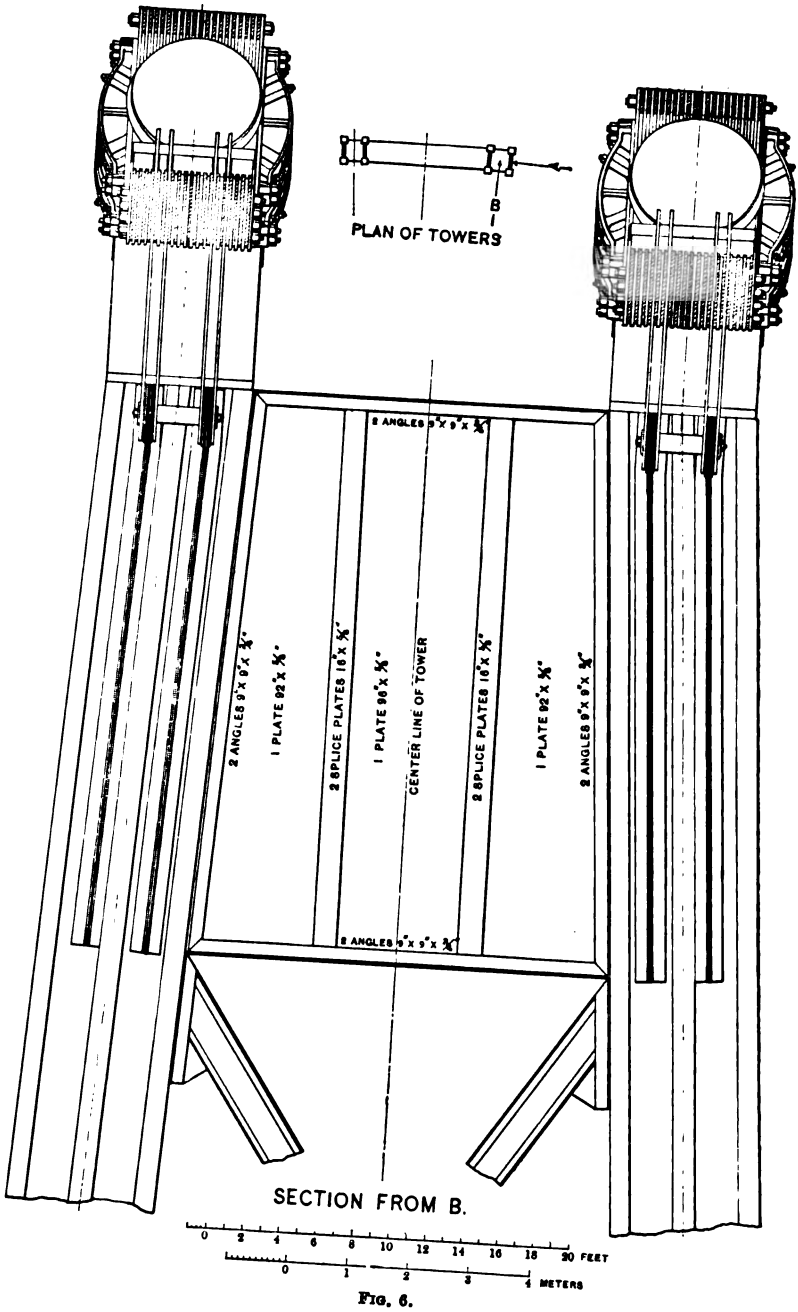

FIG. 6.

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one must be omitted so as to slip the ropes between the plates. To compensate for the omission of this pin, nineteen 10 x 2¼-in. eye-bars, coupling on a 7-in. pin, pass down a hole sunk into the rock foundation of the anchorage, and are anchored at the bottom of the hole in the manner shown in Fig. 8, these eye-bars serving only a temporary purpose. When the cables are completed, the upper pins and castings are put in position, the granite masonry built up on the castings and

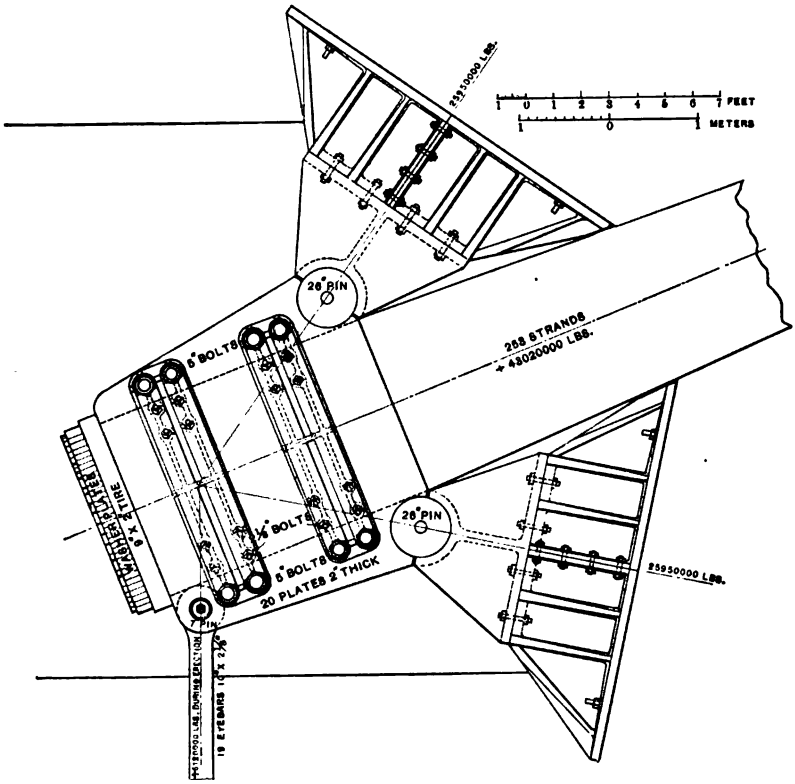

FIG. 7.

the anchorage completed. By this arrangement the cables take hold at once on the anchorage in straight lines without the intervention of eye-bars, and the strains are transmitted in the simplest possible way without the complicated curves commonly used. The cables run through tunnels sufficiently large to walk through conveniently, and the detail at the lower end is in a room about 20 ft. in each direction. The only portions of the work which will not be permanently accessible

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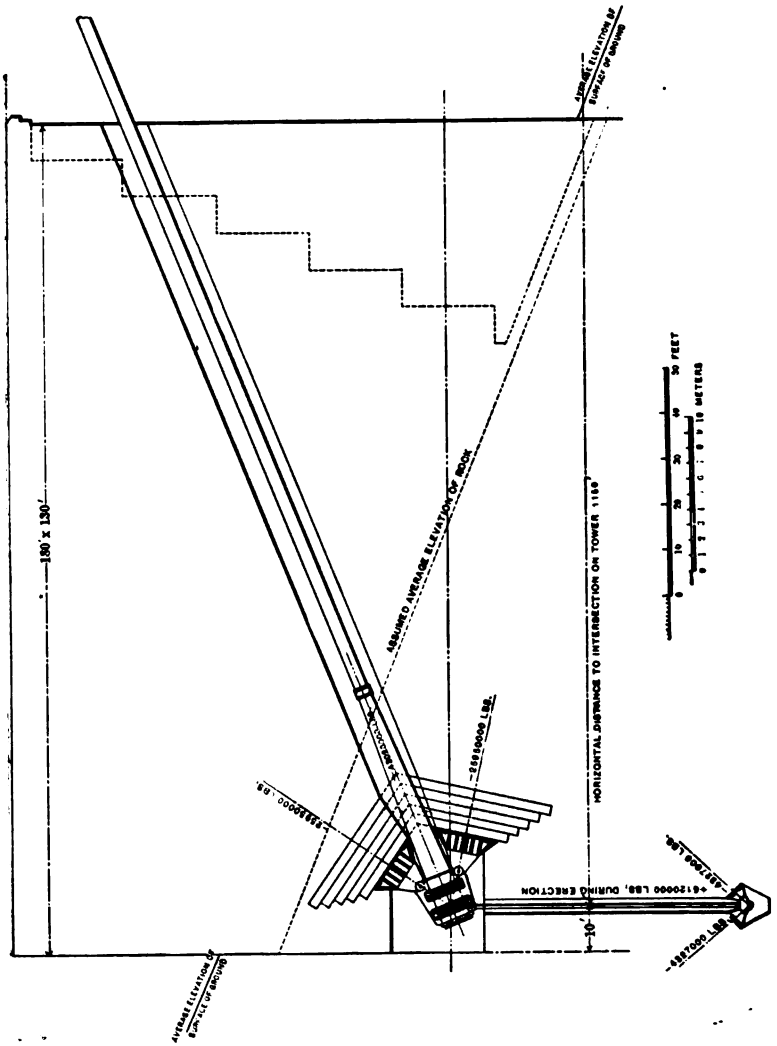

FIG. 8.

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are the eye-bars which form the lower anchorage, and these are only depended on for temporary use.

The calculations referred to above are based on the assumption that the inclination of the cables at the top of the towers is everywhere one vertical to two horizontal; in point of fact, it would be slightly flatter than this, so that the strains in the cables and the reactions on the towers are a little less than has been estimated. The actual inclinations at the top of the towers are according to the following table:

Main cable	1 in 2.196
Backstays	1 in 2.030

The actual length of the cables measured from out to out of the sockets, with an allowance for elongation under strain, is as follows:

Main cable	3 342 feet.
2 backstays, 1 268 ft. each	2 536 "
	5 878 feet.

As there are 253 ropes in each cable and four cables, the total length of rope will be 5 948 536 feet, which, at 10 lbs. per lineal foot, makes a total weight of 59 485 360 lbs. To this must be added the sockets, of which there will be 6 072; each socket will weigh 36 lbs., making the total weight of the sockets 218 592 lbs. The weight, therefore, of the ropes all socketed and ready for erection may be estimated at 59 703 952 lbs.

When the cables are completed the clamps which carry the suspenders will be put on. The form of clamp proposed is shown in Fig. 9, and is quite unlike that commonly used in suspension bridges. The clamp is formed of two steel plates pressed into shape and bound together by eight steel bolts; the lower half is a perfectly plain steel plate, but the upper half has two auxiliary plates riveted on, to hold the saddles which carry the suspenders. By means of cast-iron fillers the irregularities in the cables are filled out, and the whole is then surrounded by a sheet of thin metal about 6 ins. longer than the clamp plates, this thin metal being simply for protection against weather. The clamp-plates are then put on and screwed up tight so that the full friction which can be produced by the bolts is obtained. Two

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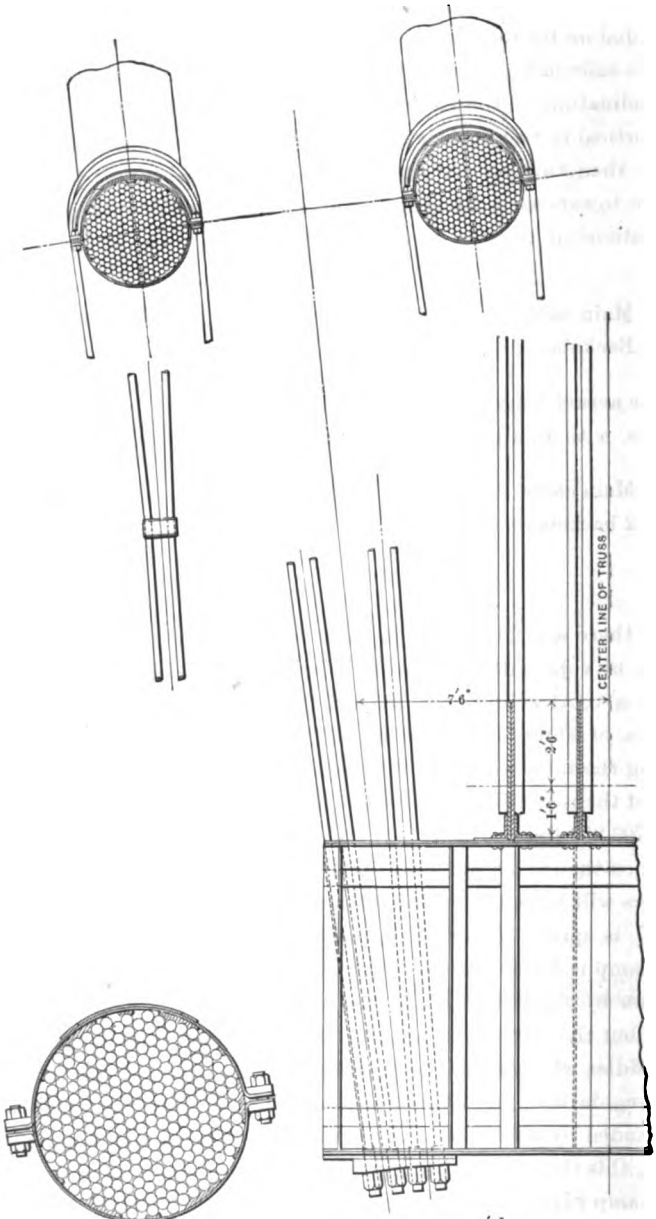

Fig. 9.

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bent saddles of soft metal are then placed on top of the clamp and everything is in readiness to attach the suspenders. Each clamp complete weighs 1 800 lbs., and there are 84 clamps on each cable, making 336 in all. There will also be clamps of different form, but about the same weight at the points where the separation of the ropes begins, 50 ft. from the end of each cable, 24 clamps being required for this purpose. The total number of clamps will, therefore, be 360, the estimated weight of which is 648 000 lbs. The weight of cables and clamps complete will, therefore, be 60 351 952 lbs.

It is thought best not to wrap cables of this size, made of independent ropes, with soft wire as is usually done, but to cover them with a thin layer of some non-conducting substance, which will allow the heat from the sun's rays to reach the metal of the cables no faster than it will be dissipated through the whole volume of the cable. It is premature to say just what the constitution of this non-conducting covering should be; it would probably be finished with a painted canvas surface, and it is not likely to weigh more than 50 lbs. per lineal foot of cable. It would extend from clamp to clamp, covering the projecting ends of the thin metal under the clamps. It must be remembered that it can be easily removed and repaired at any time.

As the cables are first made they will hang in parallel curves between the towers, and as the backstays diverge from the towers there will be a slight transverse tension between the tops of the towers; this, however, is not enough to require any special provision.

The next work, which can be done before the covering is put on the cables, is to cradle them. This cradling will be accomplished by tying the cables on each side together by steel tension bars of varying length, and tying the pairs of cables together by cross-ropes at intervals. These cross-ties and ropes would be attached to every fifth clamp, and the stress in them is comparatively low, amounting to only 90 000 lbs. each, so that ropes of 2½ ins. diameter will be sufficiently large. There will be twelve of these ropes in all, and the total estimated weight of these ropes, together with the ties between cables, is 23 000 lbs.

As soon as the cables are cradled and tied up, everything will be in readiness to attach the suspenders.

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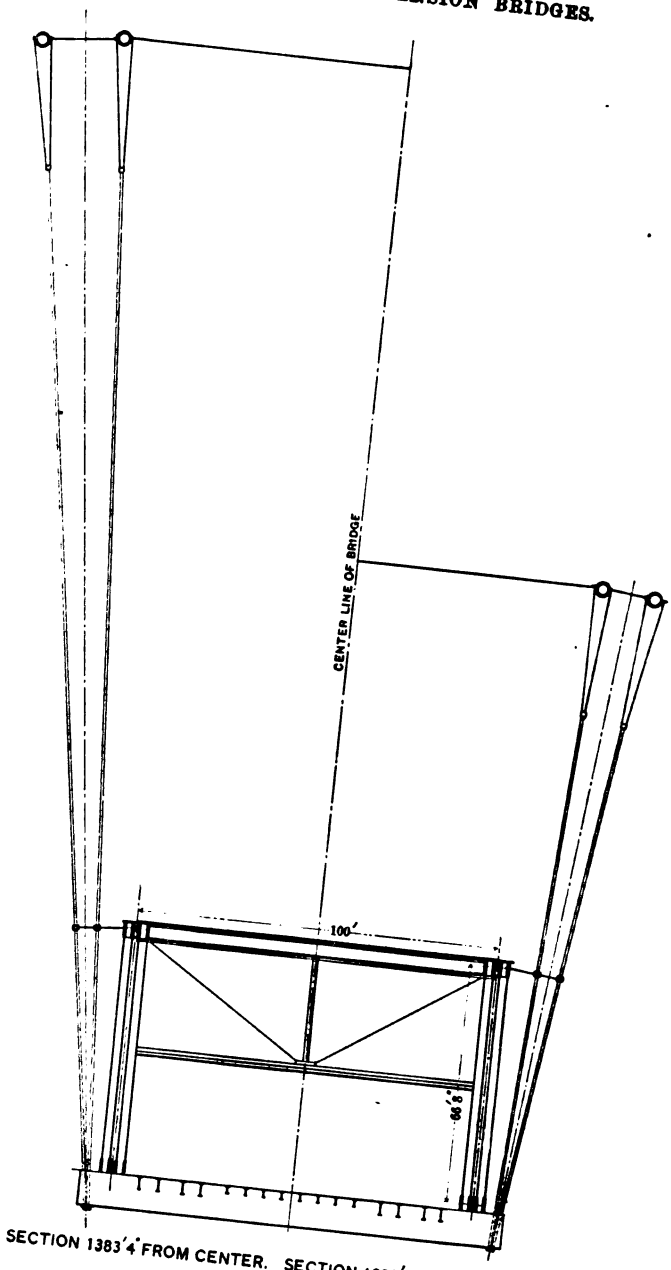

Fig. 10.

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The total weight of each main cable between vertical intersection points may be taken as follows:

3 325 ft. of cable, at 2 530 lbs. per foot....	8 412 250	lbs.
84 clamps, at 1 800 lbs.....	151 200	"
Cradling-ties.....	5 750	"
	8 569 200	lbs.
3 000 ft. of covering, at 50 lbs.....	150 000	"
Total.....	8 719 200	lbs.

This divided by 3 200 gives 2 725 lbs. as the average weight per foot of bridge for each cable and connections, or 10 900 lbs. for the four cables.

As the total weight for which the bridge is proportioned is 50 000 lbs. per lineal foot, or 12 500 lbs. for each cable, the weight which must be carried by the suspenders will be 39 100 lbs. per lineal foot for the four cables or 9 775 lbs. for each cable. There are three suspenders to each 100 ft., so that the weight to be carried by each separate suspender is 325 833 lbs.

The arrangement of the suspenders is shown in Figs. 9, 10 and 11. They are wire ropes of the same character and dimensions that are used in the main cables. The detail selected provides for four suspending ropes at each clamp. Each rope would therefore have to carry 81 458 lbs., equivalent to 27 153 lbs. per square inch, or less than half the stress allowed in the main cables.

There are really but two ropes used at each point, each rope being twice the length of the suspender and fitted at each end into a long socket on which a screw is cut. Each rope passes over the saddle and so forms two suspending ropes; the long sockets pass through washer plates under the floor-beams and are adjusted by nuts under these washers, a detail which might be modified in construction. The suspenders are clamped together about 36 ft. below the cables so as to prevent unnecessary vibration, and where it can conveniently be done the cables will be connected with the stiffening trusses.

The estimated weight of the suspenders, including the sockets, is 1 699 500 lbs., to which may be added 30 500 lbs. to provide for the small vibration connections and extras, making the total weight of suspenders 1 730 000 lbs. or 618 lbs. per foot for 2 800 ft.

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The wind strains are transferred to the towers where the stiffening truss passes the towers by cables. There will be sixteen of these cables in all, and the estimated weight of these sixteen cables, including sockets, is 36 650 lbs. The details of this arrangement, which is very simple, will be explained hereafter.

The total weight, therefore, of the rope-work in the bridge, including sockets, will be as follows:

Main cables, as above	60 374 952 lbs.
Suspenders and connections	1 730 000 "
Wind cables	36 650 "
Total	62 141 602 lbs.

These finished ropes can be furnished at the bridge site at a price from 5½ to 6 cents a pound, and the cost of placing them ought not to exceed one-half cent a pound. In these estimates the cost of these ropes has been figured at 7 cents a pound, erected, this including both main cables and all other rope-work, which makes the total cost of cables, suspenders and other similar matters, $4 349 912.

The special details which form the cable connections at the tops of the towers, and in the anchorage, are shown plainly in Figs. 2, 7 and 8. The cable connection on top of each tower-post weighs 454 461 lbs. As there are sixteen posts, the weight of these top connections, including the steel castings, is 7 271 376 lbs.

The connections at the foot of each backstay, including castings and temporary anchorage in the rock, weigh 645 451 lbs., and as there are eight of these, the total weight will be 5 163 608 lbs.

The total weight, therefore, of all material in the special details by which the cables are connected, both at the top of the towers and at the bottom of the anchorage, will be as follows:

Details at top of tower	7 271 376 lbs.
" bottom of anchorage	5 163 608 "
Total	12 434 984 lbs.

This work is generally heavy and simple, but there is in it some nice machine work, and it will probably be expedient to make all the castings of steel; it is, therefore, estimated at 5 cents per pound in position. This makes the cost of these details $621 749.

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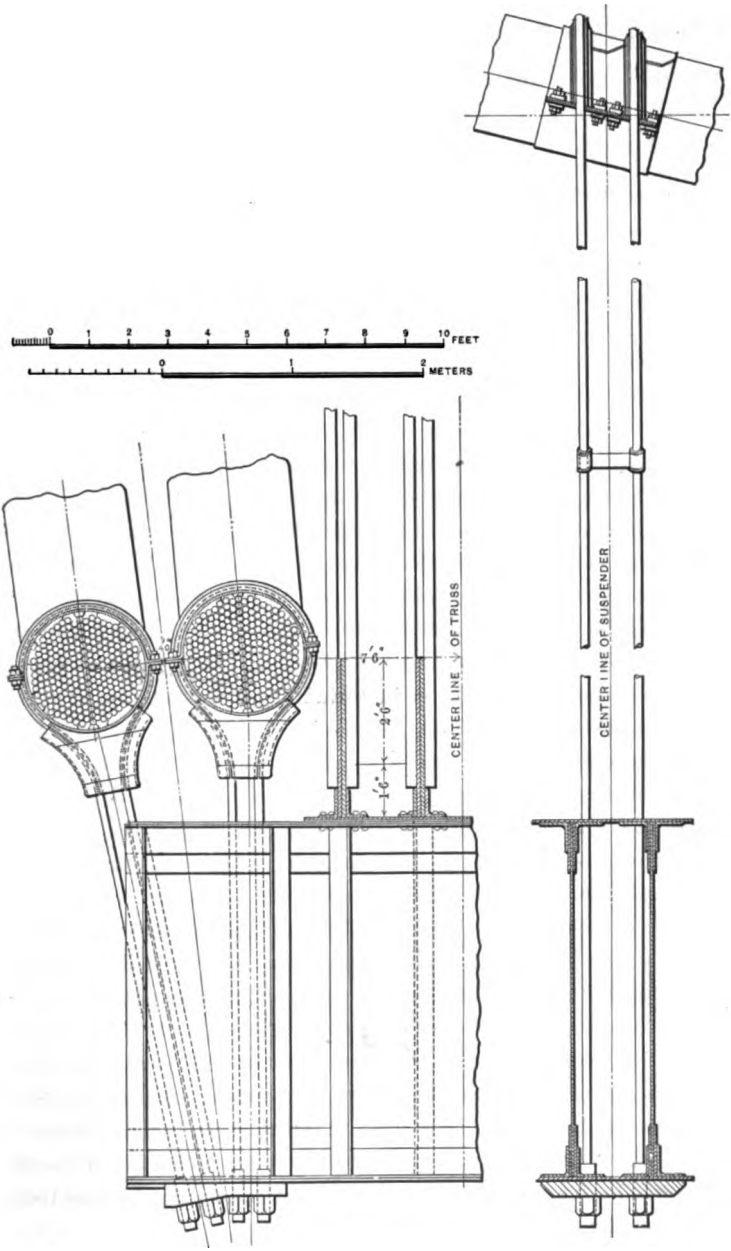

SUSPENDERS AT CENTER

FIG. 11.

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The cost, therefore, of the cables, including all wire rope and connections, will be as follows:

Rope work, etc.	$4 349 912
Connections at towers and anchorage	621 749
22 000 ft. cable covering, at $1	22 000
Total	$4 993 661

Towers.

The towers naturally follow the cables in studying the design. The support of the cables at each end of the main span consists of two towers, which form a double tower. Each tower is of approximately square section, with four corner posts, each battering one in sixteen in both directions.

In designing these towers the special functions which they have to perform must be considered. The arrangement by which the cables are attached to the top of the towers holds the towers absolutely, there being no movable saddles. Any change of length in the backstays must be taken up by a change in the position of the top of the tower. These movements at the top of the tower, combined with changes in length in the main cables, regulate the position of the suspended superstructure. It is important that the towers should be comparatively slender, so that they can bend without overstraining the metal. As the top of the tower is anchored by the backstays, a broad base is not necessary for stability.

The tower is not exactly square, but on the top the north and south sides are in line with the backstays, and the distance between theoretical intersections is 28 ft. on each side. As, however, the outer cables are lower than the inner cables, the actual distance between centers at the bottom of the castings, or at elevation 559.08, will be 29.18 ft. and 29.55 ft. for the north and south sides and 28.42 ft. and 27.58 ft. for the east and west sides of each tower. At the theoretical intersection point between the bottom strut and the posts at elevation 36, the north and south sides of the tower measure 92.94 and 93.8 ft. and the east and west 93.93 and 94.32 ft. Although the tower is not exactly square, it is so nearly so that the irregularity would seldom be observed. The towers, however, are twisted with reference to each other, and this would be manifest.

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Strain sheets have been prepared for these towers, and are given in Figs. 12 and 13, these strain sheets showing the calculated results from the 20 000 000 lbs. imposed on the top of each post, and from the weight of the tower itself, beside the strains due to wind. The towers have been proportioned on the basis of a stress of 20 000 lbs. per square inch from weight alone in the posts, no additional provision being made to resist wind or other extraordinary strains, as they will in no event be more than 25% greater than the strains produced by weight alone. The assumption supposes the cables to have an inclination of two horizontal to one vertical. The real strains in these towers are, therefore, from 1% to 10% less than the calculated strains.

The actual motion in the top of the tower after the completion of work, due to changes in the length of the backstays caused by a maximum moving load, on the basis of a modulus of elasticity of 26 000 000, will be 6⅞ ins., which corresponds to a stress of 2 176 lbs. per square inch in the posts of the tower. This is less than 11% of the 20 000-lb. stress for which the posts of these towers are proportioned; as the towers have been proportioned on the supposition that the angle of the main cable is two horizontal to one vertical, whereas it is 10% greater, and the reaction on the posts 10% less, the stresses in the posts, after allowing for bending, are only 1% more than the calculated strain. The motion at the top of the posts, caused by a change of temperature of 60°, will be less that 5½ ins., corresponding to a stress of 1 450 lbs. per square inch in the posts, which may exist in either direction from a mean.

During erection a much greater motion may be required, especially if no temporary supports are put in for the backstays. A greater motion can be obtained without overstraining the metal, in two ways: by omitting the permanent diagonal bracing and putting in timber bracing, which can be so wedged as to bend the tower; by inserting hydraulic presses in the castings under the posts next to the river, so that the whole tower can be thrown out of plumb.

The section of each post varies from 1 051 sq. ins. at the top to 1 145 sq. ins. at the bottom. Each post is 8 ft. square, and the details of construction are shown in Fig. 14. This post is divided into quarters and made in sections, each 24 ft. long, breaking joints with these sections, so that there would be one horizontal quarter joint every 6 ft. Each quarter section 24 ft. long would weigh less than 12 tons, a

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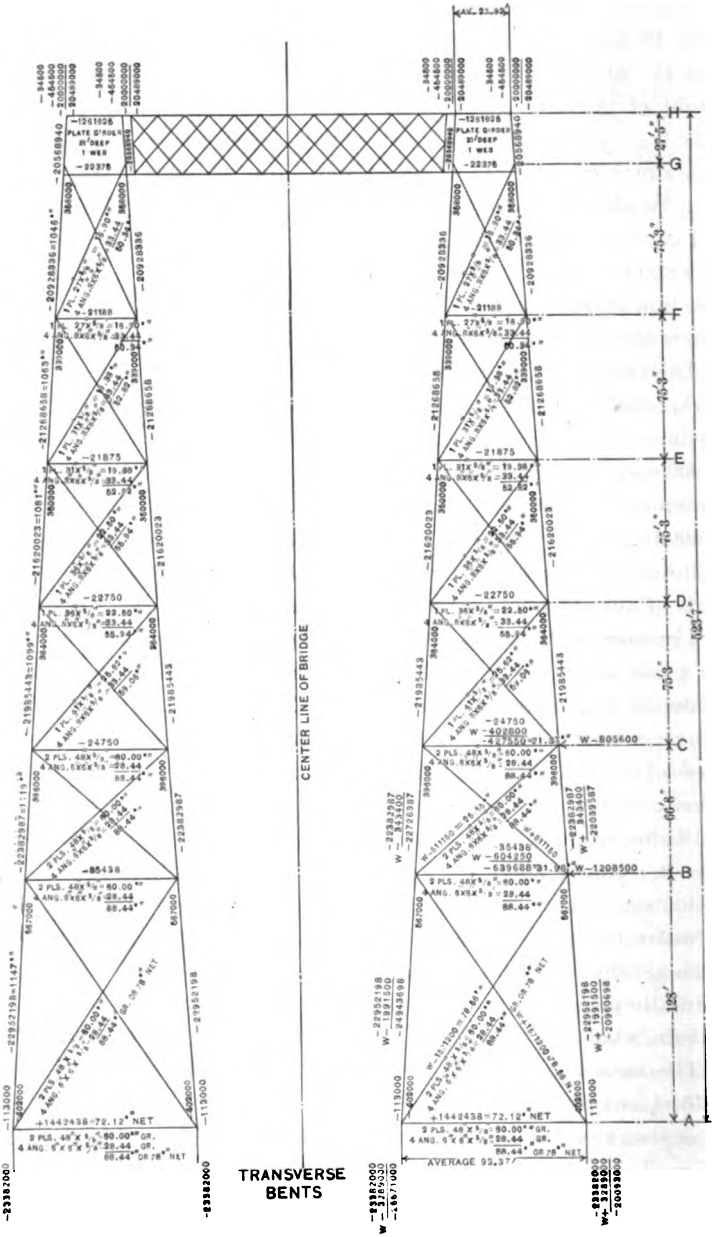

FIG. 12.

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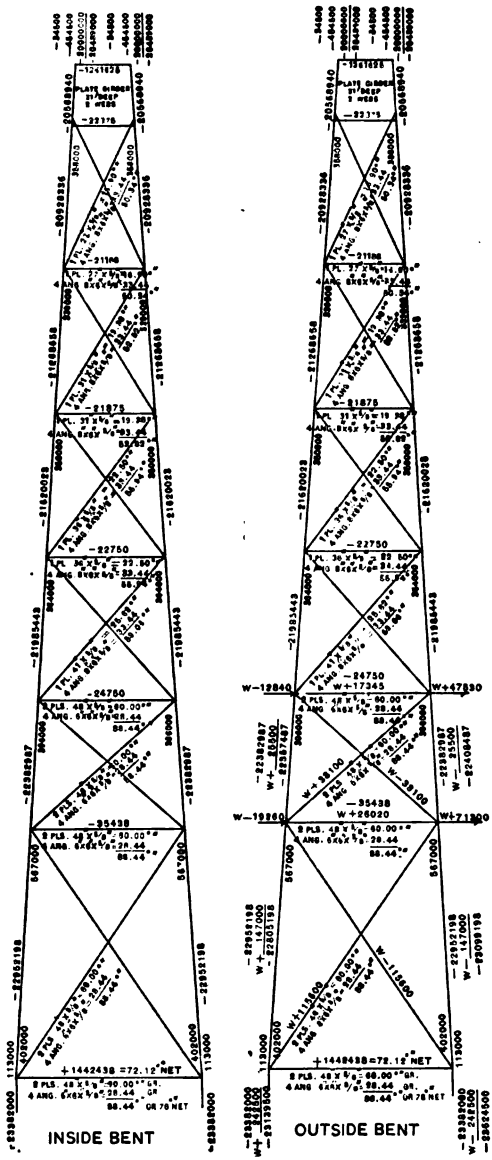

FIG. 13.

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weight which can easily be handled. All the heavy riveting around the corners would be shop driven, and it is proposed to use 1¼-in. rivets for this portion of the work. The splices at the joints, both vertical and horizontal, would be field driven, ⅞-in. rivets being used in these places, and could, if necessary, be hand driven. At intervals of 24 ft. diaphragms would be built in each post, these coming opposite one of the joints (as shown in the intermediate section in Fig. 14), the function

BILL FOR ONE-QUARTER POST SECTION.

1 Plate, 48" x 1⅛", 24' long. 1 Plate, 21" x 1 1/16" to 1⅛", 24' long.
1 " 46 5/16" x 1⅛", 24' long. 1 Angle, 9" x 9" x 1", 24' long.
1 " 22" x 1 1/16" to 1⅛", 24' long. 1 Corner Plate, 25" x ⅞", 24' long.
2 Splice Plates, 15" x ⅞", 24' long.

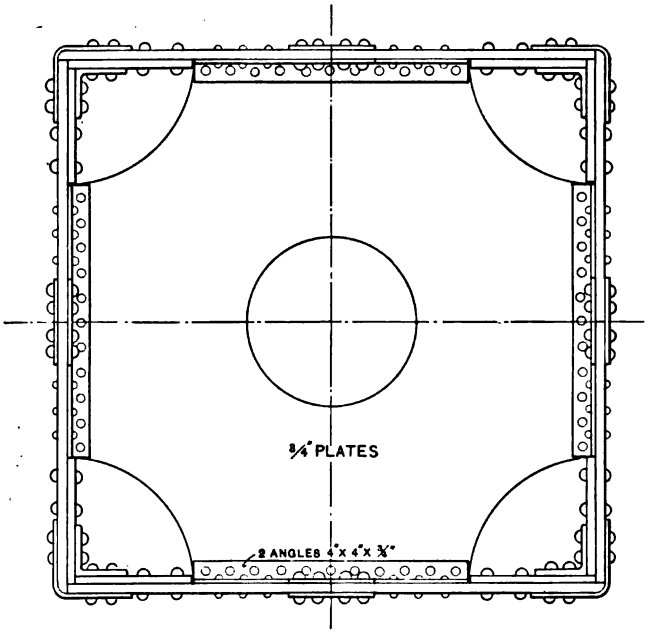

FIG. 14.

of these diaphragms being the same as that of the diaphragms in a bamboo rod. At the top two extra cross webs would be built into the post to support the steel casting.

At the bottom the post would rest on a large casting. For convenience of inspection, a hole is made through the middle of each diaphragm, and a series of ladders would reach from diaphragm to diaphragm, by which inspectors could pass through the whole interior

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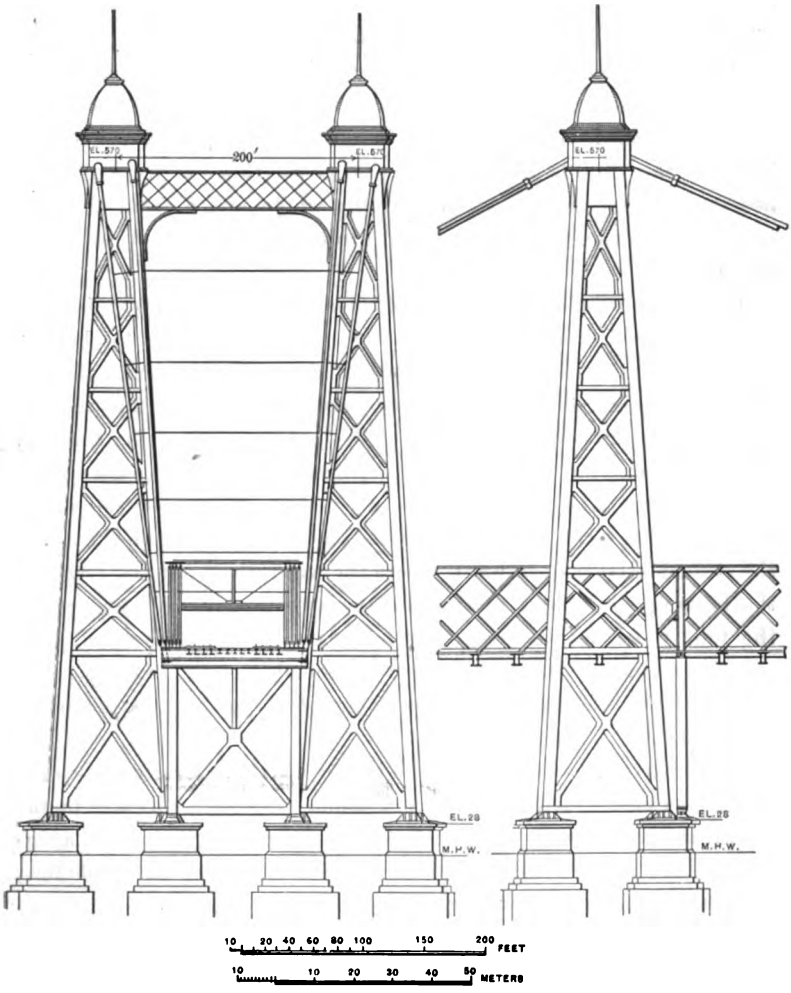

FIG. 15.

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of the post, a manhole being placed near the base, through which they could enter. At the bottom each post would be held down by an anchor bolt at each corner, though this is hardly a necessary provision.

These posts have been carefully estimated in detail, the weight of one post being 2 182 000 lbs., or 8 728 000 lbs. for the four, including diaphragms, ladders and everything else.

At the top the four posts are connected by girders 31 ft. deep, there being two girders on each longitudinal side and one on each transverse side. The depth of these girders is fixed by the riveted connection between the double girders and the posts, this connection being necessarily long enough to transfer the whole strain from the vertical bars to the post. These six girders are estimated to weigh 264 000 lbs.

The tower is braced on each side between the four posts, this bracing being divided into six panels, the second panel from the bottom corresponding in height to the depth of the stiffening truss, this arrangement being adopted so that the wind strains can be thrown from the stiffening truss into the tower at the panel points of the bracing. The arrangement of this bracing is given in Fig. 15. Above the stiffening truss the bracing has comparatively little to do, and is made in the form of a single web with broad angles on the edges. From the top of the stiffening truss downward, where the wind strain is to be resisted, the bracing is double webbed and materially heavier. At the bottom the four posts are tied together by heavy riveted ties, which pass outside the posts, but form the bottom member of the bracing. The weight of the bracing for one tower complete has been estimated at 2 389 000 lbs.

The towers are connected at the top by a light lattice truss bridge, which has comparatively little work to do, but which will add to the lateral stability, and be a convenience, both during erection and afterward. This truss bridge complete is estimated to weigh 98 000 lbs., or 49 000 for each single tower.

Each post rests on a large bottom casting, which should be made of steel. This casting would be made in four principal parts bolted together, and should be machined to a true plane on the bottom, as the pressure on the bottom will be 1 000 lbs. per square inch. This casting, together with the anchor bolts, is estimated to weigh 130 000 lbs., or 520 000 lbs. for each tower.

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The total weight of the structural portions of the tower will then be as follows:

Posts	8 728 000 lbs.
Girders	264 000 "
Bracing	2 389 000 "
Truss	49 000 "
Castings and anchor bolts	520 000 "
Total	11 950 000 lbs.

As there will be four of these towers, two at each end, the total weight of metal in the towers will be 47 800 000 lbs. This entire work should be open-hearth steel of the quality ordinarily used for structural purposes. The sections are so heavy that the edges would all have to be planed, and it would all be solid drilled work; special machinery would be required for its manufacture, but the order is so large that the cost of this special machinery when averaged over the whole work would be no greater than the wear of ordinary tools. In open competition this work ought to be let for a very low price per pound; if estimated at 4 cents per pound, there results for the cost of the towers $1 912 000.

Fig. 15, showing the general elevation and tower, shows the tower as finished with a room about 50 ft. square on top, surmounted by ornamental work terminating in a flagstaff. This covered room is necessary to house the cable connections, but as the floor of this room will be 560 ft. above the water, or nearly twice as high as any structure now existing in New York City, and higher than any structure in the world except the Eiffel Tower, a considerable revenue could probably be derived by providing elevators and taking visitors to the top of the towers. As the elevators and housings on top of the towers do not form a portion of the structural work, they are not included in the estimate.

Foundations.

The average depth from mean high water to rock at the site of each tower is assumed to be 140 ft. In order to prevent any possible disturbance from expansion and contraction of transverse members at the feet of the metallic towers, it is thought best to rest each post on an entirely independent foundation. There would, therefore, be four independent foundations under each tower, or eight on each side of the

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river, making sixteen in all. Of these the two center ones next to the river, on each side, would have to support the reaction of the stiffening truss as well as the weight of the tower post, and they are therefore made larger than the others. The plan of the tower foundations is shown in Fig. 16.

Each separate foundation consists of a masonry pier of granite 58 ft. high, finishing 28 ft. above mean high water, and terminating at the base at an elevation 30 ft. below mean high water. It is proposed to make these piers of an exceptionally good class of masonry and to allow a maximum pressure of 20 tons per square foot on the granite, and of 1 000 lbs. per square inch on the under side of the casting which

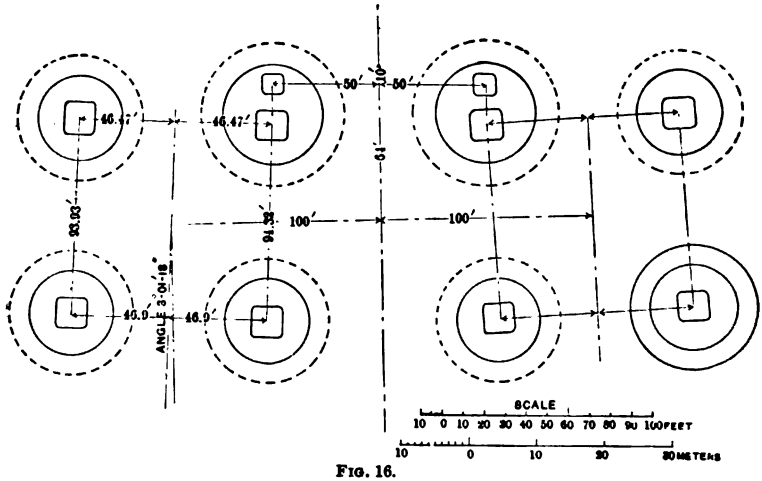

FIG. 16.

supports the tower. While these pressures may seem excessive, it must be remembered that good granite has a crushing strength of over 20 000 lbs. per square inch, and that the pressure in the central section of the East River Bridge towers, at the level of the roadway, is 28 tons per square foot. The masonry pier for each of the smaller foundations contains 2 700 cu. yds., and that for the larger foundations 3 900 cu. yds. Estimating this work at $30 per cubic yard, the masonry for the smaller foundations becomes worth $81 000, and for the larger foundation, $117 000.

Each of the smaller foundations would rest on a cylinder 60 ft. in diameter and 110 ft. high, thus containing 311 000 cu. ft., or, say,

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11 500 cu. yds. Each of the larger foundations would rest on a cylinder 70 ft. in diameter, thus containing 423 330 cu. ft., or, say, 15 700 cu. yds. If the cost of this portion of the work is estimated at $20 per cubic yard, the cost of each of the smaller foundations becomes $230 000, and that of each larger foundation $314 000.

The cost of each of the smaller piers complete, including both masonry and foundation, becomes $311 000, and of each of the larger piers, including both masonry and foundations, $431 000. As there are at each end of the bridge six of the smaller piers and two of the larger, the cost of the substructures of each of the double towers will be $2 728 000. The total cost of the foundations for the towers on both sides of the river will then be $5 456 000.

This estimate is believed to be at least $1 000 000 more than the actual probable cost of such work, but this paper is dealing with the general subject of a suspension bridge, and precise estimates of the subaqueous portion really pertain to more specific subjects.

The pressure on the bottom of the foundations after deducting the weight of the material displaced, and estimating the weight of a cubic foot of masonry or foundation at 150 lbs. in air, 87 lbs. in water, or 50 lbs. in mud, is 9.304 tons per square foot for the smaller, and 9.306 tons for the larger.

Anchorages.

The anchorages at each end of the bridge would be divided into two parts, each of which anchors two cables, the position of these anchorages being shown in Fig. 1.

The anchorage has no duty to perform except to provide weight, and may be built of a very cheap class of masonry or of concrete. Any class of work which is entirely free from voids and weighs at least 140 lbs. per cubic foot, or 3 780 lbs. per cubic yard, will answer this purpose. The exposed sides of the anchorages should be faced with a good class of masonry; brick would answer, but granite would be more in keeping with the massiveness of the work. The top will not require a coping, but should be covered with Portland cement concrete or some form of pavement which will keep out water; there is no reason why buildings should not be erected on top of the anchorages.

There will be two tunnels running through each anchorage, each of which should be lined with brick, and be large enough for con-

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venient inspection of cables, and perhaps, also, for running a carrier during erection. At the lower end of each cable there will be a room in which the detail connection is placed, and it will probably be expedient to have some kind of a staircase placed in a small shaft by which these two rooms can be reached. The bearing of the castings must be taken on granite masonry of very high quality, the pressure on the bottom of the castings being 1 000 lbs. per square inch, and enough of this masonry has been provided to reduce the pressure on the cheap masonry to 250 lbs. per square inch.

Each anchorage would consist of a single block of masonry, a longitudinal section of which is given in Fig. 8. It is 180 ft. long, 130 ft. wide, and the top finishes at elevation 155, this being the elevation of the rails. The horizontal distance from the theoretical intersection on the top of the tower to the theoretical intersection at the bottom of the anchorage is taken as 1 150 ft. The lower intersection is assumed to be at elevation 60, so that the vertical difference in height between the two intersections is 510 ft.

The volume of material above the theoretical intersection point is 2 223 000 cu. ft., which at 140 lbs. per cubic foot, weighs 311 220 000 lbs. The angle at which the cables take hold of the anchorage is one vertical to 2.524 horizontal. The vertical lift will, therefore, be 31 695 721 lbs. Taking this from the weight of the anchorage, the weight left to resist the horizontal pull is 279 524 279 lbs. Assuming a coefficient of friction of 60 per cent., the frictional resistance of this weight is 167 714 567 lbs. Dividing this last quantity by 80 000 000 lbs., the assumed horizontal strain in cables, gives a factor of stability of 2.09. This is without taking into consideration the strength of the additional anchorage in the rock below, nor including the weight of any buildings or other structures which may be erected on top of the anchorages.

The quantity of masonry in each anchorage above the points of intersection is 2 223 000 cu. ft., or 82 333 cu. yds. As, however, the foundation may be below the assumed level of the intersection, this masonry is estimated as 100 000 cu. yds. The greater part of this would be concrete or a cheap class of masonry, but 1 375 cu. yds. will be high-class granite work. Estimating the whole mass of masonry at $6 per cubic yard, and adding $44 per cubic yard (making a total cost of $50) for the high-class granite work, there results as the cost

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of one anchorage $660 500. There will be two anchorages at each end of the bridge costing $1 321 000; the total cost of the anchorages at both ends of the bridge will, therefore, be $2 642 000.

Suspended Superstructure.

The suspended superstructure embraces the floor beams and the stiffening truss, with all the necessary cross-bracing, laterals, etc. The stiffening truss is the principal feature of the whole, and its peculiar function is such that the calculation of the exact strains is a work of extreme difficulty. It is possible, however, in an investigation of strains to bring them without extreme difficulty within limits which they surely will not pass in either direction, and as everything is based on elastic changes in which there must always be a small percentage of irregularity, these results are at least as accurate as the material is uniform, and the error will always be on the safe side.

A stiffening truss with a hinge at the center has the advantage of greater simplicity in the calculations, but the details of the hinge are much more objectionable than any irregularities of strain which might occur, and a continuous stiffening truss without a hinge has been used in this design.

The functions of a stiffening truss may be considered in two ways. It may be regarded simply as a floor stiffener, preventing short local changes; or it may be considered as a complete stiffening truss which distributes the entire moving load with practical uniformity over the whole length of the structure. The former is the usual function performed by the stiffening truss of a long-span highway bridge; the latter is the function which a stiffening truss must perform in a short-span bridge or in a railroad bridge of moderate length.

In the former case the proportioning of the stiffening truss is comparatively simple. The greatest possible distortion of the cables under a moving load must be calculated, the moving load being considered uniform for such distance as will produce the greatest distortion; the amount of deflection which will occur within the limits of this load must then be determined, and the stiffening truss made of such depth that it can deflect this amount without overstraining the metal in the chords; as the deeper the truss, the greater the unit stress in the chords for the same deflection, it follows that the stresses must in this instance be kept down by using a shallow truss. This works

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well in highway bridges; it would work equally well in a railroad bridge of such dimensions that the dead load would be so great in proportion to live load that the deflection would not exceed the limit over which trains could conveniently be run at high speeds.

In the case of a railroad bridge in which the dead load is light in proportion to the moving load the stiffening truss must be proportioned on such a basis that it will virtually distribute the whole moving load uniformly over the entire length of span. As the deflection of the stiffening truss has little influence in such calculations, the deeper the stiffening truss, the less will be the stresses in the chords.

In the present case the dead load is so great in proportion to the moving load that the distortion of the cables will be comparatively small, even under the passage of trains; it will, however, be so great that if the stiffening truss performed no other functions than that of a floor stiffener, the deflection might disturb the rapid passage of trains. In this case, the proper method of proportioning a stiffening truss is, to decide first what deflection will be allowable under maximum conditions; then to determine what depth of stiffening truss will correspond to the maximum unit stresses which it is considered wise to adopt; then to determine the amount of difference in load which will be required to create a corresponding distortion in the cables; by deducting this difference from the total moving load, the amount of moving load is determined which the stiffening truss must distribute. On this basis a stiffening truss may be proportioned, though the exact strains must be a matter of subsequent computation.

The condition of loading which will cause the greatest deflection in the loaded portion of the stiffening truss will occur when the maximum moving load covers one-half of the 2 800 suspended feet of stiffening truss, occupying 1 400 ft. on either side of the center; this is also the case in which all calculations are most simple. A limit of deflection of one four-hundredths of the half span, or 3½ ft. in 1 400 ft., corresponds to a 1% grade at each extreme end of the deflection, and has been selected as the limit in this case.

The moving load which the cables are capable of carrying is equivalent to 11 000 lbs. per lineal foot over the entire structure, and it is assumed that this load is distributed over the equivalent of six railroad tracks, corresponding to 1 833 lbs. per foot

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of track. In estimating the effects of an unequal load a weight of 12 000 lbs. per lineal foot is taken in accordance with the provisions stated at the beginning of this paper. While it may be considered that the load per foot on one-half or one-third of a span ought to be more than 10% greater than the load per foot on the whole span, it must be remembered that the peculiar conditions of this bridge are such that it is only under very rare conditions that any considerable portion of the moving load must be distributed by the stiffening truss. In fact two passenger trains could cross this bridge side by side without disturbing the position of the cables beyond the limits of deflection which are permissible; it is only when the load exceeds that of two maximum passenger trains that the stiffening truss has any duties to perform beyond that of a floor stiffener.

The excess load required to produce a distortion of 3.5 ft. at the quarter on a 2 800-ft. span with a versed sine of 310 ft. (which corresponds to the design), will be 9.424% of the load on the unloaded portion. Taking the dead load at 39 000 lbs. per lineal foot, 9.424% of this is 3 675 lbs.; deducting this from 12 000 lbs., there remains 8 325 lbs. as the weight per foot to be distributed by the stiffening trusses, or 4 162 lbs. for each truss.

By the use of nickel steel, as hereafter explained, considerably greater strains may be used in the stiffening truss than would be considered good practice with ordinary structural steel; a stress of 17 000 lbs. per square inch has therefore been selected as the limit. This having been assumed, the depth of the stiffening truss may be calculated by the following formula:

$$d = \frac{5\ S\ l^2}{24\ E\ v_o}$$

in which

S = stress per square inch = 17 000
E = modulus of elasticity = 30 000 000
v_o = deflection = 3.5
l = span = 1 400

Solving this quotation, $d = 66.11$.

For convenience the depth has been made 66 ft. 8 ins., equal to $200 \div 3$.

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The required section of the chords of the stiffening truss will then be determined by the following formula:

$$a = \frac{\frac{w}{2} l^2}{8\, d\, s}$$

in which

$$d = 200 \div 3$$
$$= 1\,400$$
$$s = 17\,000$$
$$w = 4\,162$$

Solving this equation, $a = 450$.

In the design the chords of the stiffening truss have a gross section of 600 sq. ins. and a net section of 560 sq. ins.; the net section is 24% greater and the gross section 33% greater than the approximate calculation requires; it is expedient, however, to provide an excess of metal above that required by these advance calculations. The gross section is the one to be used in calculating deflections. Should the modulus of elasticity assumed (30 000 000) be criticised as being too high, it must be noted that this modulus is applied to the calculated section of the chord, whereas the actual section would be materially increased by splices and connection plates, so that a modulus of 27 500 000 in the metal would correspond to at least 30 000 000 in the calculated section.

This is a simple solution and gives data to start from, but while considering the effect of the distorted cables in carrying unequal loading, it does not consider the bending effects on the stiffening truss due to the deflection of the cables as a whole, all of which must be taken into consideration in the final calculations. In these final calculations it is necessary to assume certain sections of cables and stiffening truss chords as well as loads. It is not possible to make a strain sheet in advance and then proportion the sections in accordance with the strains. Everything is determined by deflections, and deflections are themselves determined by sections.

A careful investigation of the theory of the stiffening truss has been made on entirely independent lines by Mr. Charles S. Peirce. By the use of the formulas which this investigation has developed, the strains in the stiffening truss have been calculated for three different conditions. In the first of these a moving load of one-third the length

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of the span, 3 100 ft., is assumed to occupy the first third of the 3 100 ft. In the second supposition it is assumed to occupy the second and third sixths of the 3 100 ft., the end of the load being at the center. In the third supposition it is supposed to occupy the central third of the 3 100 ft. The moments developed by these calculations are a satisfactory confirmation of the capacity of the stiffening truss.

In this connection it is interesting to compare the duty which is to be performed with that on the East River Bridge. The stiffening

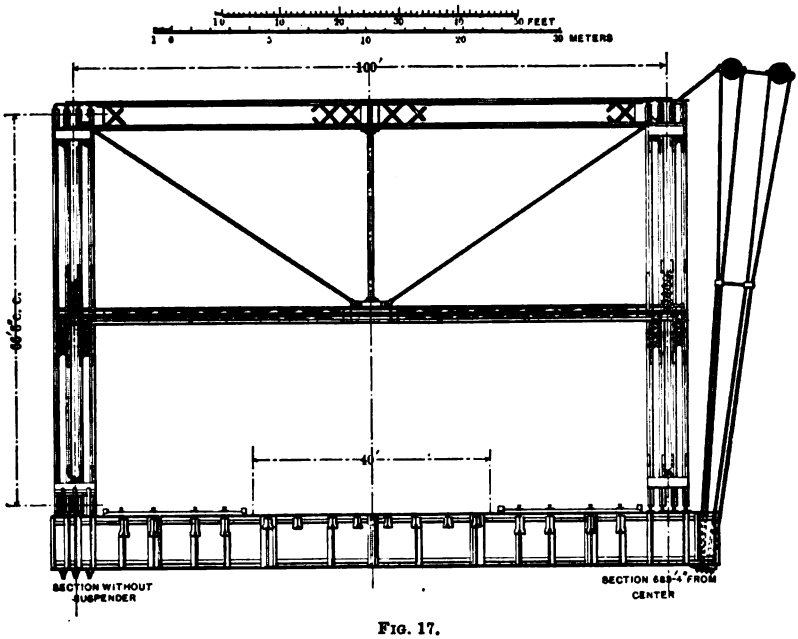

FIG. 17.

truss of the East River Bridge is simply a floor stiffener; both chords are cut at the centers of both the main and the side spans, having sliding joints which can transfer no bending strains whatever and a comparatively small amount of shear. The trains which run over this bridge weigh about 100 tons each, or half a ton per lineal foot, about two-thirds the weight of a first-class passenger train, and they are about half as long as an ordinary passenger train; and yet, in spite of the lightness and theoretical inadequacy of the stiffening truss, the action of the bridge under these trains is entirely satisfactory.

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In all these calculations no account has been taken of the stiffness of the cables themselves, which are more than 3 ft. in diameter, of the chords of the stiffening truss, which are 4 ft. deep, nor of the stringers of the floor system, all of which would have decided influences of their own.

The two stiffening trusses designed are each 66.67 ft. deep between centers of gravity, or 70 ft. over all. They are placed 100 ft. between centers. There is a stiff riveted lateral bracing between the top chords and a transverse bracing, as shown in Fig. 17, at every panel point. The floor system is entirely below the bottom chord and the bottom laterals are built in as a portion of this floor system. The webs are riveted lattices with four independent lines of bracing.

The suspended portion of the truss is carried by the floor beams, and as its weight exceeds the amount of moving load which has to be distributed, its action really amounts to varying the portion of its own weight which is transferred to the floor beams, there never being any conditions under which any portion of the weight of the floor system has to be transferred to the stiffening truss. Beyond the limits of the suspenders the floor beams are hung from the stiffening truss to which they transfer the weight of the floor system.

The arrangement of the floor system as worked up is shown in a general way in Fig. 18. The floor beams are strong enough to carry the whole weight from truss to truss, thus leaving a clear space for the whole distance. The two double-track railroads are placed next to the trusses, thus reducing the weight of the floor beams to a minimum, while the possible deflection of one end of the beam below the other is found not to be enough to produce trouble.

There will be eight railroad stringers in each panel and eight lighter stringers which carry the roadway or the rapid transit tracks. Each railroad stringer is proportioned for a total load of 4 230 lbs. per foot of stringer, the strains in the extreme fiber being 9 025 lbs. per square inch of gross section, and weighs 6 300 lbs. Each light stringer is proportioned for a total load of 1 000 lbs. per foot of stringer, the strain in the extreme fiber being 9 620 lbs. per square inch of gross section, and weighs 2 540 lbs.

The floor beams are proportioned to carry a weight of 45 667 lbs. at each single railroad stringer connection, and for a load of 100 lbs. per square foot on the 40 ft. between railroad tracks, besides a weight of

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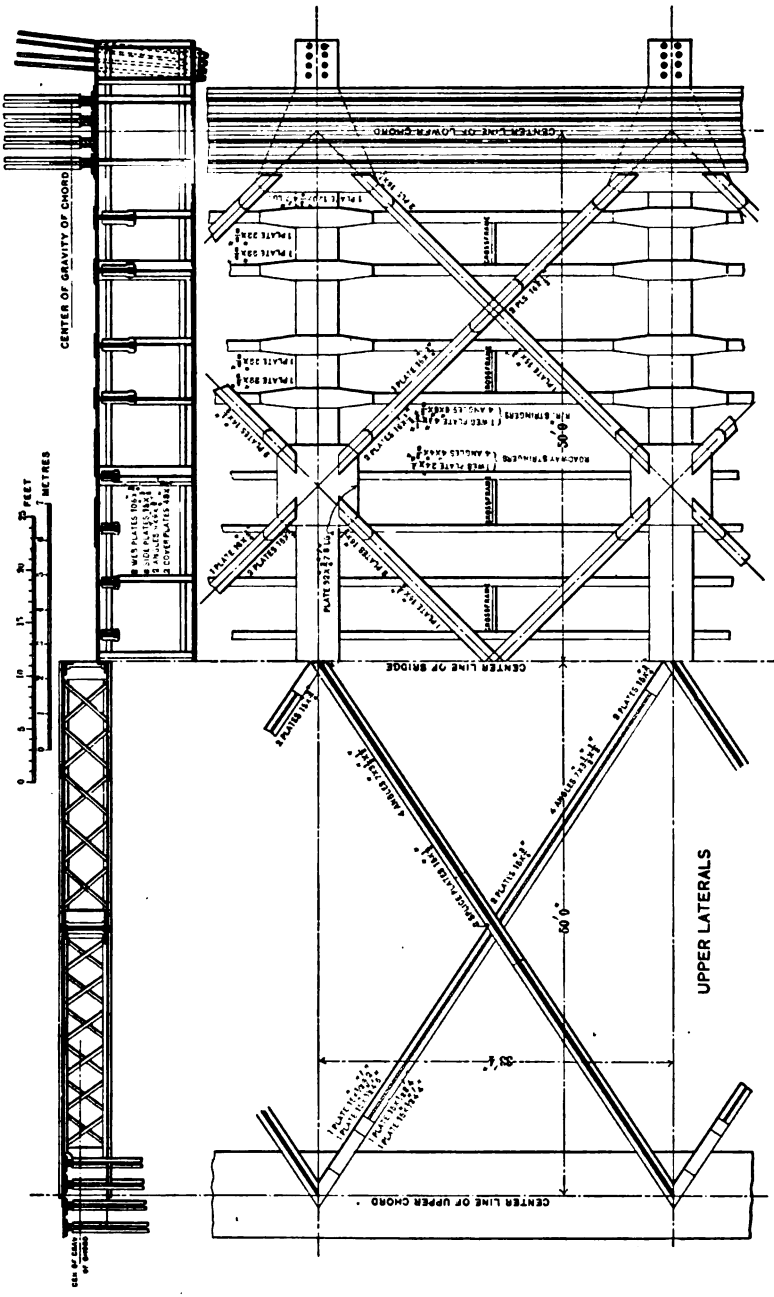

FIG. 18.

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280 000 lbs. at each end from the stiffening truss. Under these conditions the strain in extreme fibers of gross section is 12 938 lbs. Each floor beam is really a box girder with two webs, and weighs 179 000 lbs. The total weight of the metallic floor system is therefore as follows:

	Per panel.	Per foot.
Floor beams	179 000 lbs.	5 370 lbs.
Railroad stringers	50 400 "	1 512 "
Rapid transit stringers	20 320 "	610 "
Lower laterals and connections	24 880 "	746 "
Total	274 600 "	8 238 "

The upper laterals and transverse bracing together weigh 48 000 lbs. per panel, making the total weight suspended, exclusive of the weight of the stiffening trusses themselves, 322 600 lbs. per panel, or 9 678 lbs. per lineal foot of bridge.

Each chord of the stiffening truss has, as has been already stated, a gross section of 600 sq. ins., and the weight of each chord per lineal foot, including splices and connections, is estimated at 11 100 lbs. Each of these chords is made with four webs, the details being as shown in Fig. 17. The arrangement of the webs of the stiffening truss is also shown in Fig. 17. The webs are made uniform for the entire suspended portion of the stiffening truss. Each web member has a total gross section of 53 sq. ins. and a net section of 41 sq. ins., and under the extreme conditions which have been specified above, namely, a moving load of 8 325 lbs. per foot of bridge, exclusive of that which is distributed by the distortion of the cable, these members are subjected to a strain of 7 855 lbs. per square inch of net section, and 6 373 lbs. per square inch of gross section, these strains occurring in the same member in both directions. The web as so designed weighs 3 240 lbs. per lineal foot.

The dead weight of floor, including ties, rails and other work, has been assumed at 3 000 lbs. per lineal foot of bridge, being 500 lbs. for each railroad track and 1 000 lbs. for the intermediate 40 ft.

The lower laterals act only in tension, and their weight, as estimated, includes a large amount of detail connections.

The top laterals and transverse bracing are determined by minimum sections for the most part instead of by strains. The upper laterals

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weigh 750 lbs. per lineal foot, and the transverse bracing 690 lbs. The total weight, therefore, of the suspended superstructure per lineal foot may be taken as follows:

Metallic floor as above	8 238 lbs.
Stiffening truss chords	11 100 "
Stiffening truss webs	3 240 "
Upper laterals and cross-bracing	1 440 "
Total metal work	24 018 "
Tracks and flooring	3 000 "
Total	27 018 "

The total dead load is therefore as follows:

Cables and connections (page 493)	10 900 lbs.
Suspenders and connections (page 493)	618 "
Suspended superstructure	27 018 "
	38 536 "
Add for telegraph line and sundries	464 "
Total	39 000 "

This leaves 11 000 lbs. of the total 50 000 lbs. available for moving load, as already stated, which may fairly be considered a margin of 2 000 lbs. over anything that is likely ever to occur.

The total length of the stiffening truss from out to out, including the 500-ft. spans at each end, is 4 100 ft. Assuming chords and floor system to be uniform throughout, the weight of this 4 100 ft., taken at 24 000 lbs. per lineal foot, will be 98 400 000 lbs.

This, however, includes the weight of the heavy floor beams within the suspended length, while there are four floor beams entirely omitted at the supporting points and thirty-six floor beams which are themselves suspended from the chords. Neglecting one end floor beam, as the estimate has been made on a basis per lineal foot, and assuming that the floor beams hung from the chords are 30 000 lbs. lighter than the others, there is a deduction on this account as follows:

3 floor beams at 179 000 lbs	537 000 lbs.
36 floor beams reduced, at 30 000	1 080 000 "
	1 617 000 "

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On the other hand, the webs near the supporting bents will have to be reinforced, which can be done by making the members of greater width than elsewhere. As there are no reverse strains here, it is thought right to fix the limit of stress at 20 000 lbs. per square inch of net section. On this basis the additional metal required in the webs is 3 418 000 lbs.

To this must be added the weight of four vertical posts over the rocking bents, and four vertical posts and two portals at the ends of the continuous superstructure.

The weight of the entire suspended superstructure will then be as follows:

4 100 feet (page 515)	98 400 000 lbs.
Additional metal in web	3 418 000 "
Vertical posts over rocking bents	545 000 "
End posts	467 000 "
Portals	140 000 "
Total	102 970 000 "
Deduct for floor beams	1 620 000 "
	101 350 000 "

At the center of the bridge the cables are so little above the floor beams that the stiffening truss must be considered as fastened to the cables longitudinally at this point, a condition which is assumed in the refined calculations of strains ; as it is continuous it must be free to move longitudinally at all other points, and especially at the ends. As the continuous truss is 4 100 feet long, a motion due to the effect of temperature on 2 050 ft. must be provided for at each end, this being 0.82 ft. for 60° of temperature; a possible motion of 1.64 ft. must therefore be provided at each end of the stiffening truss, or 0.82 ft. from either side of a mean. This motion is too great to be accommodated by roller bearings of the ordinary kind, and the design places the stiffening truss on rocking bents, which are shown in Figs. 15 and 19. The possible reaction at points 3 100 ft. apart is assumed to be 10 736-000 lbs., and this is taken on vertical posts, the pair of posts under the two trusses being braced together, thus forming a rocking bent, which is supported on two of the cylinders which form the tower foundations. These special cylinders have therefore to sustain this

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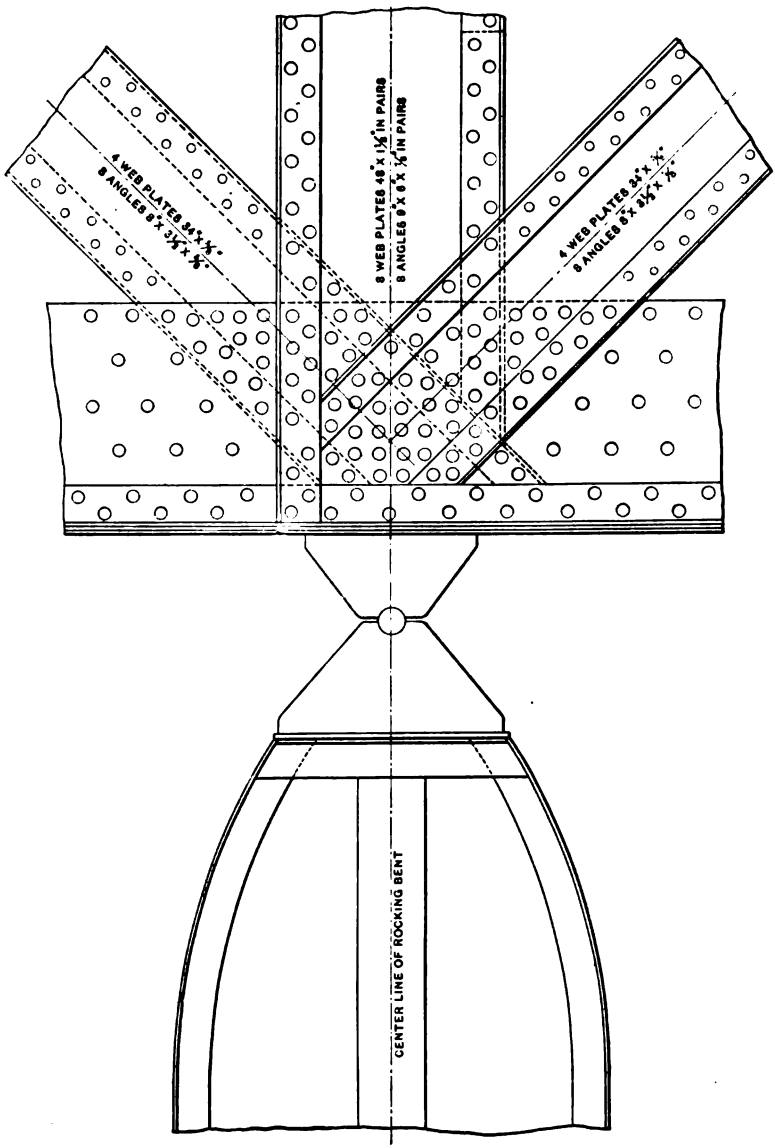

Fig. 19.

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weight in addition to the weight received from the tower, and for this purpose their size has been increased, as has already been stated, and the center of the foundation has been placed at a point between the two bearings, the distance of which from each bearing is inversely proportional to the respective reactions.

At each end the stiffening truss rests on a rocking bent of smaller dimensions, which rests on a masonry pier.

Each rocking bent at the principal point of support is estimated to weigh, complete, 1 005 000 lbs., and each rocking bent at the extreme ends 480 000 lbs., making, as the total weight of the four rocking bents, 2 970 000 lbs. If to this is added the weight given above, the total weight of the metal work in the suspended superstructure and connections becomes 104 320 000 lbs. This work would all be of structural steel, and in it the strains will nowhere exceed 20 000 lbs. per square inch of gross section under actions of load alone.

The strains in the chords are reversed, and these are further increased by the bending which occurs in the truss under the rise and fall of the cables from the effects of temperature and by the effect of wind. The estimated rise and fall at the center from changes of temperature is 3.3 ft., which, calculated on the same formula as used previously and taking $l = 3\ 100$, corresponds to a strain of 3 330 lbs. per square inch.

The bending strain due to the deflection of cables under weight has not been considered in this connection, but in the more refined calculations is considered in connection with all other changes of shape caused by weight in determining the moments on the stiffening truss.

The strains are subject to reversal, and represent, including the effects of temperature, a possible variation of 40 000 lbs. per square inch between extreme positive and negative strains, or 20 000 lbs. in each direction ; this is higher than it is deemed wise to place on ordinary structural steel, and requires a material which, while possessing the toughness of the soft steel preferred for structural purposes, has the strength and high elastic limit of the harder steels. Five years ago such a material would have been considered impossible ; it may now be found in nickel steel containing about 3½% of nickel, a material which will have an elastic limit of about 60 000 pounds per square inch and can be subjected to the reverse strains just referred

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to, and under extreme occasional conditions could be worked to at least 40 000 lbs. per square inch without injury. As nickel steel is a comparatively new article, made by a few manufacturers, though it has been adopted to a very great extent by the United States Government in its naval work, it is difficult to learn just what the additional cost ought to be; apparently it is worth, on the basis of cost, about three-quarters of a cent more per pound than ordinary structural steel, but it has been estimated as costing 2 cents per pound extra, this representing additional mill and shop work, though the latter is very little.

The modulus of elasticity of nickel steel is practically the same as that of ordinary structural steel, and it is proposed to use it only for the principal members of the chords for a length of 2 000 ft. at the center of the bridge, where reversals and wind strains are large. The rivets, splice plates, etc., need not be of nickel steel. The weight of nickel steel in each chord may therefore be taken at 2 000 lbs. per foot, or 8 000 pounds for the four chords, or 16 000 000 lbs. for the 2 000 ft.

The work in the stiffening truss is of a very uniform character, and, considering its great weight, ought to be obtained at a very moderate price per pound; it is estimated at 4 cents per pound, with an extra allowance for nickel steel.

The total cost of the 4 100-ft. stiffening truss, supporting bents, etc., may therefore be taken as follows:

104 320 000 lbs. at 4 cents	$4 172 800
16 000 000 " nickel steel, at 2 cents extra	320 000
Total	$4 492 800

Shore Piers.

To sustain the ends of the stiffening truss two additional piers will be required. These piers should be founded on rock, but would be piers of ordinary dimensions, and, though large, would present no special difficulties of construction. On the plans, Fig. 1, they are shown with the masonry finishing at an elevation 60 ft. above mean high water, which is probably higher than necessary, and the depth to rock is assumed to be 80 ft. below mean high water. The piers are assumed to be 20 ft. wide and 120 ft. long on top, the masonry to start

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at the water level and to be founded on a caisson and surmounted by a timber crib filled with concrete, the whole foundation being 35 ft. wide, 135 ft. long and assumed as 80 ft. high. The cost of these piers is estimated as follows:

6 000 cu. yds. masonry at $25	$150 000
378 000 cu. ft. foundation at 60 cents.	226 800
Total	$376 800

The cost of the two piers, one at each end, would then be $753 600.

Wind Pressure.

The wind surface per lineal foot presented by one-half of one web, the lower chord and the floor system is 11.35 sq. ft., and the wind surface presented by the upper half of the web and the upper chord is 7.77 sq. ft. As the trusses are 100 ft. apart, the area of the trusses should be doubled, but the floor comes so near to being solid that it need not be doubled. The total surface presented to the wind which must be resisted by the top laterals is therefore 15.54 sq. ft. per lineal foot, and the total surface presented to the wind which must be resisted by the bottom laterals is 19.12 sq. ft. per lineal foot. To the latter should be added the area of a passing train, which is equivalent to 8 ft. above the bottom chord, thus making the total wind surface to be provided for 27.12 sq. ft. per lineal foot. On a basis of 30 lbs. per square foot the total wind pressure to be resisted is—

Top lateral system	466 lbs.
Bottom lateral system	814 "
Total	1 280 "

For the calculations, these figures have been slightly varied, and the top laterals are proportioned to resist a wind pressure of 500 lbs. per lineal foot and the bottom laterals a wind pressure of 750 lbs. per lineal foot.

There is no probability that anything like these wind strains will ever be reached over the whole length of the span, though considerably greater pressures may occur for limited lengths. To reduce these amounts, however, would be a departure from established practice.

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The wind pressure would be transferred to the towers where the stiffening truss passes the towers, by horizontal cables, these cables reaching from each chord to the outer posts of the towers, the cables clearing the inner posts and being long enough to provide for the longitudinal motion of the trusses without overstraining. These horizontal cables would be tightened under strain so that they would always stiffen the trusses. A portion of the wind strain would undoubtedly be taken by the transverse bracing of the rocking bent. Furthermore, the continuity of the truss beyond the rocking bent would reduce the equivalent length of the central span. In calculations this reduction has been assumed to be about 50 ft. at each end, though this is undoubtedly much less than it would really be. On this basis the bending strain produced by wind in the bottom chords will be—

$$\frac{3\,000^2 \times 750}{8 \times 100} = 8\,437\,500$$

This corresponds to about 14 000 lbs. per square inch on the 600 sq. ins. of the bottom chord, and gives a deflection calculated as above of 8.75 ft. Should this occur when there is a maximum strain in the chords from the passage of trains, a condition which would probably not take place more than once in a century, the chords might possibly be strained to 34 000 lbs. per square inch. With nickel steel this is perfectly safe.

In these calculations it is assumed that the chords of the stiffening truss are the only longitudinal members, which is by no means correct, as the sixteen stringers will act as auxiliary chords in the wind system.

There is also another element which materially reduces the effect of wind. To produce the above-mentioned strains in the chords, the whole suspended superstructure must move laterally 8.75 ft. This involves swinging the main cradled cables and raising the center of gravity of the suspended superstructure, a lateral movement of 8.75 ft. corresponding to a lift of 0.075 ft. or 1 vertical to 117 horizontal. As the suspended superstructure weighs 27 000 lbs. per lineal foot, this will require a horizontal force of 230 lbs., so that before this deflection can occur the actual wind pressure must be about 1 000 lbs. per lineal foot on the bottom chord.

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Rivet Strains.

While plans showing the details of riveting have only been prepared as a study, it has been necessary to form some basis on which they should be proportioned, especially as owing to the magnitude of the structure and the large relative dead loads, the unit strains vary from those ordinarily used. The simple rule has been followed that the bearing stress on each rivet should be considered equal to the stress allowed in the gross section of the member, and the shearing stress should be limited to one-half the stress allowed in the gross section of the member; this may be expressed differently by stating that the bearing surface of the rivets should not be less than the gross section of the member, and the shearing section of the rivets should be double the section of the member. Where nickel steel is used, the number of rivets is increased one-half, the bearing surface of the rivets being made 50% greater than the gross section of the member and the shearing section of the rivets three times the gross section of the member.

Erection.

The erection of the stiffening truss and suspended superstructure is a comparatively simple thing. The back spans, 500 ft. each, would be erected on falsework in the usual manner, which could be put in without difficulty as it is in a protected position back of the pierhead lines. The projections from the rocking bents to the suspenders would be built out as cantilevers, all of which could be done without trouble.

The suspended superstructure proper would be handled in a different way. The floor beams would be put in position first; they would be brought to the bridge site on barges and raised into position, each beam being hung from the suspenders as fast as raised. The stringers would be put in and riveted up as the floor beams are erected, so that when all the floor beams are up, a reasonably stiff floor would be ready to work on; this portion of the work could be done very rapidly, as each of the 84 floor beams could be handled independently.

When the floor beams are in place and the floor system riveted up it could be covered with planks and form a working platform. The bottom chords of the stiffening truss would then be put in place and riveted up, the horizontal rivets being driven by power, and the vertical rivets, which are of less importance, by hand. The only matter which would require special attention would be to see that a uniform distribution of weight was kept at all times, and this is a

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matter of discipline rather than of difficulty. As soon as the bottom chord is riveted the lower half of the webs would be erected and this would be followed with the upper half, after which the top chords would be put on and the top lateral system erected. The broad floor would form a platform on which any desirable system of travelers could be run and the opportunities for work would be as good as in a shop, except that there would be no roof.

The total weight of suspended superstructure which must be erected in suspension is about 34 000 tons. The speed with which it could be handled would depend entirely on the number of men and the amount of plant employed.

Estimate.

The work has been described in the manner in which the design has taken shape, and the cost of each separate portion has been estimated in connection with this description. In execution the work would necessarily be differently divided and may properly be grouped under the respective heads of substructure and superstructure.

Under these heads the cost may be stated as follows:

Tower foundations	$5 456 000	
Anchorages	2 642 800	
Shore piers	753 600	
Substructure		$8 852 400
Metallic steel towers	$1 912 000	
Wire work, etc	4 993 661	
Suspended superstructure, etc	4 492 800	
Superstructure		11 398 461
Total		$20 250 861

For purposes of inspection an elevator ought to be placed in each of the four towers, and two of these elevators ought to be of sufficient size to accommodate passengers; $100 000 should be reserved for these elevators and the various appliances in connection with them.

The ornamental work on top of the towers, with provisions for lighting, etc., would cost another $100 000.

The structure, with a 10% allowance for contingencies and engineering, would cost about $22 500 000, or somewhat less than $5 500 a foot for the 4 100 ft. of suspended superstructure.

By making some modifications in the plan, among which may be mentioned allowing a greater flexibility under extreme conditions and

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reducing the depth of the stiffening truss, the cost could probably be reduced to about $20 000 000.

Time.

The time it would require to construct such a bridge would depend largely on the resources of the company building it. If everything were in readiness, both legally and financially, it ought to be built in five years. The foundations for the towers could be conducted simultaneously and completed in two years. The steel towers could be erected in another year. The anchorages and shore piers could be completed before the towers are done. The cables, being already manufactured, could be erected in one year. The back spans and projecting cantilevers could be raised while the cables were being put in position. The suspended portion of the superstructure, 2 800 ft., could be erected in one year. This allows two years for the erection of the metallic towers and the placing of the cables, which could probably be materially reduced. Five years would therefore appear to be enough for the construction of this bridge.

APPENDIX A.* TEST OF WIRE ROPES, MADE AT U. S. ARSENAL, WATERTOWN, MASS., MAY 2d-4th, 1895.

In the straight wire ropes the elongation was measured on 100 ins.; in the coiled wire ropes, on 200 ins.

The numbers of the tests show the order in which they were made. The general practice followed was, first to apply an initial strain of 10 000 lbs. per square inch, the recording gauge being attached after this load was applied. The strains were then increased by successive increments of 10 000 lbs. up to 60 000 lbs., when the rope was allowed to rest for at least ten minutes while a number of observations were taken. The strains were then reversed back and forth six times between 50 000 and 60 000 lbs., raised to 70 000 lbs., reversed back and forth between 60 000 and 70 000, and then gradually raised by 10 000-lb. increments until the stretch made it necessary to remove the gauge; after each increment the strain was reduced 10 000 lbs. before increasing. In a few of the tests first made, this method was not fully carried out. In every instance except those designated by stars, the percentage of elongation recorded in the table was observed after the rope had been subjected to a strain 10 000 lbs. greater than the recorded strain.

Test No. 8 275 was left under a strain of 60 000 lbs. per square inch for 16½ hours, at the end of which time this strain was reduced by

* A full report of these tests is on file in the Library of the Society.

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Test of Steel Wire Ropes.

Character of Rope.	Straight Wire.				Coiled Round Wire.				Patent Locked Wire.			
Character of wire	Special Steel.		Plow Steel.		Special Steel.		Plow Steel.		Special Steel.		Plow Steel.	
Number of test.....	8 269	8 279	8 271	8 274	8 270	8 278	8 273	8 275	8 272	8 280	8 276	8 277
Strain per Square Inch.	Percentage of Elongation above that Produced by Initial Strain of 10 000 lbs. per Square Inch.											
50 000 lbs.....	0.1525*	.....	0.1834	0.1803	0.2588	0.2752	0 2257	0.2355	0.2735	0.2325	0.2742	0.2710
60 000 "	0.1994*	.....	0.2255*	0.2195	0.3190	0.3333	0.2794	0.2817	0.3274	0.2973	0.3286	0.3259
70 000 "	0.2405*	.....	0.2711	0.2604	0.3968	0.4059	0.3351	0.3345	0.3862	0.3553	0.3937	0.3877
80 000 "	0.2889*	.....	0.3118	0.3032	0.4674	0.4811	0.3980	0.3937	0.4573	0.4229	0.4673	0.4611
90 000 "	0.3382*	.....	0.3478*	0.3476	0.5509	0.5637	0 4676	0.4575	0.5424	0.5207	0.5537	0.5479
100 000 "	0.3869*	....	0.3956	0.3930	0.5930*	0.6724	0.5345	0.5209	0.5862*	0.6007	0.6496	0.6555
Ultimate strength.....	150 000	146 640	188 980	187 360	148 120	140 000	187 230	177 690	133 180	131 970	142 220	176 850

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about 330 lbs. per square inch, when the tests were resumed; the elongations given were all measured subsequently to this rest.

Test No. 8 280 was left under a strain of 60 000 lbs. per square inch for 39 hours, during which time the strain was reduced about 5 000 lbs. per square inch, when the tests were resumed; the elongations given were all measured subsequently to this rest.

Each of the round wire ropes was formed of 37 No. 8 wires. Each of the locked ropes was formed of 62 wires, of which the central one was round, the intermediate wires square and the outer layer of special lock section. The straight wire ropes were wrapped with fine soft wire.

In the cases of the two ropes which were left under strain, one over night and the other over two nights and an intermediate Sunday, the strains were reversed back and forth between 50 000 and 60 000 lbs. several times when testing was resumed, and the observations under these conditions were specially interesting and valuable. They were as follows:

Strain per Square Inch.	Elongation in 200 Ins.	
	Number of Test, 8 275.	Number of Test, 8 280.
60 000 lbs.	0.5510	0.5852
50 000 "	0.4711	0.5051
60 000 "	0.5511	0.5853
50 000 "	0.4711	0.5051
60 000 "	0.5513	0.5851
50 000 "	0.4711	0.5051
60 000 "	0.5512	0.5852

These show an extraordinarily uniform modulus of elasticity of 25 000 000 lbs., and show how uniformly this rope may be depended upon for action in a structure, even though there be a material difference in the quality of the wires. These observations were made prior to those recorded in the preceding table. The elongation for the 50 000-lb. strain is the same in the two tables. The difference between the elongations for the 60 000-lb. strains in the two tables is due to the set taken when the strain was raised to 70 000 lbs. before the 60 000-lb. strain was observed for the first table.

Five wires were taken from two of the straight wire cables, one being of special steel and the other of plow steel. The samples taken from the special steel showed an average strength of 172 588 lbs. per square inch, and an average reduction of 44.2 per cent.; the plow steel showed an average strength of 226 504 pounds per square inch and an average reduction of 45.7 per cent. In the case of the plow steel one wire was nicked before testing; its strength was fully up to the average of the others, but its reduction was so much less that it has been excluded in calculating the average reduction.

The fractures always occurred first in the outer wires and the ropes evidently failed to develop their full strength owing to defects in sockets, which were not as well finished inside as they should have been.

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DISCUSSION.

T. C. CLARKE, M. Am. Soc. C. E.—This paper is both interesting and instructive. It shows what to do, and what not to do. Mr. Clarke.

The system of socketed connections devised by the author has enabled him to plan the best system of anchorage known to the speaker. It has the great merit of enabling all the parts to be accessible for inspection and painting. On the other hand, the severing of the cables at the top of the towers seems unnecessary. It is true that it gives shorter and lighter parts to handle, but if a strand 3 342 ft. long can be handled, one 5 698 ft. long can also be handled. Its weight, taken at 10 lbs. per ft., is 28½ tons, which is not greater than that of street railway cables which have been transported by railway cars and wagons.

The adoption of a unit made of a number of wires instead of a unit of one wire is good, and will save time and interest during construction. Engineers may differ as to the plan of a twisted wire rope, but all must agree that this Society owes its thanks to the author for having made and reported his interesting experiments at the Watertown Arsenal.

The design of the towers is excellent in every respect. They give the greatest possible strength with the least material.

The speaker strongly objected to a stiff suspended girder, either with or without a hinge in the center. Such a design tries to unite two systems which are always working against each other, and the connections of the suspended girders, being the weakest parts, must give way sooner or later. The author's girder has reversal strains of 40 000 lbs. per inch in all, and he admits that ordinary steel will not do.

The only possible way to insure the durability of a suspended girder is to treat it merely as a distributor of concentrated loads, as Roebling did in his bridges. He purposely made his girders shallow and flexible. A flexible girder, however, is unsuited to the heavy trains and rapid speeds required by railway traffic. A better plan is to brace between the cables, as Gustav Lindenthal, M. Am. Soc. C. E., has done in his design. For such long and heavy spans, it is possible to utilize the cables themselves as chords without excess of material. Such a plan is more economical than the deep and rigid suspended girder. Engineers do not seem to realize how great the economy of this braced cable construction is. A comparison between that part of the designs of Mr. Lindenthal and the author will show this clearly.

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Comparison of Designs for Suspension Bridges, Hudson River at New York.

Description.	Engineer Commission. August 23d, 1894.	G. S. Morison, Paper. October 4th, 1896	G. Lindenthal. July 20th, 1894.	W. Hildenbrand. July 20th, 1894.	Board of U. S. Engineer Officers. October 25th, 1894.
Length of land span	965	1 000	1 850	500	900
" of main span	3 220	3 200	3 100	3 310	3 200
" of land span	965	1 000	1 850	500	900
Total length between anchorages	5 150	5 200	6 800	4 310	5 000
Number tracks and load per foot	6 x 3 000 lbs.	8 x 1 500 lbs.	8 x 3 000 lbs.	6 x 3 000 lbs.	6 x 3 000 lbs.
Total load per foot	18 000 "	12 000 "	24 000 "	18 000 "	18 000 "
Strain per square inch on wire of 180 000 lbs., ultimate strength	60 000 "	58 921 "	65 000 "	50 000 "	60 000 "
Reversal strains suspended girder	12 500 x 2 lbs.	20 000 x 2 lbs.	?	15 000 x 2 lbs.	?
Tons wire work	42 104	30 160	57 800	42 230	28 159
Do. per lineal foot	8.17	5.80	8.50	9.80	5.63
Tons suspension girder	50 565	52 160	37 260	28 110	26 156
		4 100			
Do. per lineal foot	9.80	10	5.5	6 7	5.23
Do. suspended	92 569	82 320	95 060	71 040	84 315
Do. per lineal foot	18	16	14	16.5	10.86
Total tons cables, girders, towers, anchorages, etc.	145 438	110 950	125 700	118 940	108 912
Tons per lineal foot	28.24	21.3	18.5	27.6	21.78
Cost, superstructure	$16 086 111	$11 270 949	$11 072 850	$12 079 700	$10 402 540
" substructure	17 480 960	8 852 400	9 927 200	3 830 000	11 784 000
" miscellaneous		2 375 756			
" total	$33,517 671	$22 500 00	$21 000 000	$15 900 700	$22 186 540
Interest, 5% for five years on half above amount	4 189 625	2 812 500	2 225 00	1 968 625	2 773 317
Total cost	$37 707 296	$25 312 500	$23 225 000	$17 890 325	$24 959 857
Cost per lineal foot between anchorages	7 321	4 870	3 415	4 151	4 990

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Mr. Clarke.

DESIGN BY G. S. MORISON.

Cables	6.13 tons per foot at $140	$858	
Suspended girder	12.70 " " " " 86	1 092	
Total	18.83 " " "	1 950	
		$1 950 × 4 100 =	$7 995 000

DESIGN BY G. LINDENTHAL.

Cables and bracing	8.5 tons per foot at $140	$1 190	
Floor stiffener, girder	5.5 " " " " 80	440	
Total	14.0 " " "	1 630	
		$1 630 × 4 100 =	6 683 000
	Difference		$1 312 000

When it is considered that Mr. Lindenthal's cables carry a moving load of 12 tons per foot, or double that of Mr. Morison's cables, which carry 6 tons only, the economy of the braced cable system becomes evident.

As it is an open secret that Mr. Morison's design was made for the crossing of the Hudson River at New York, a comparison between it and the designs which were submitted to and by the Engineer Commission and the Board of U. S. Engineer Officers in 1894 will be both interesting and instructive. The table on the preceding page is instructive as showing how experts can differ in working out the results of the same problem.

If the moving load taken by Mr. Lindenthal were reduced from 12 tons to 9 tons per lineal foot as assumed to be sufficient by the Engineer Commission, the saving of live load would be 19 800 tons. This would not only reduce the weight and cost of the cables, but also that of girders, towers, anchorages and foundations, and the whole saving would be at least 15 per cent. on the cost of bridge.

Mr. Lindenthal estimates the cost of his bridge,

if 5 500 ft. long, at	$17 800 000
deduct 15%	2 670 000
	$15 130 000
Add interest for five years at 5% on half the cost	1 891 000
	$17 021 000

This is less than half the sum estimated by the Commission of Engineers; but a careful examination of Mr. Lindenthal's quantities and prices fails to discover any serious error.

If this estimate is correct, it means that the cost of a six-track suspension bridge over the Hudson River at New York, between Sixtieth and Seventy-second Streets, would not exceed the probable cost of a cantilever bridge designed to carry 3 000 lbs. live load on each of six tracks, having a central span of 1 600 ft. and two side spans of 800 ft. each, supported by two piers in the river. Mr. Lindenthal deserves the credit of having been the first to demonstrate this by a plan which, if built, would be a work of real architectural merit.

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Mr. Mayer. JOSEPH MAYER, M. Am. Soc. C. E.—The design for a suspension bridge made by the author differs in several important points from that of the Union Bridge Company for a suspension bridge of the same span accommodating six railroad tracks. The Union Bridge Company's bridge has twelve cables lying in vertical planes, and two suspended stiffening trusses in each half span, hinged at the center and ends. The stiffening trusses have curved top chords, and are 200 ft. high in the center; they do not project beyond the towers. In the construction of the cables, the Union Bridge Company followed American precedents. In the author's design the cables are cradled, the stiffening trusses are continuous at the towers and in the center, they have parallel chords, and are only 66 ft. 8 ins. high.

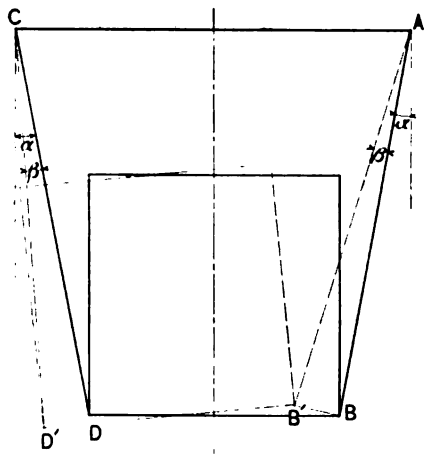

FIG. 20.

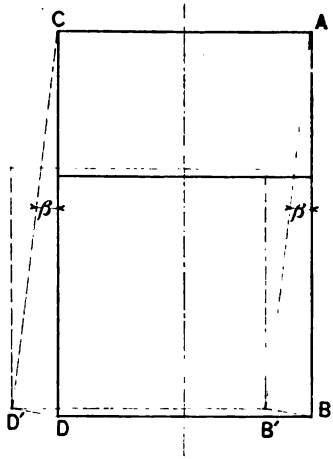

FIG. 21.

In the arguments brought forward in favor of the cradling of cables it is always shown how stable against lateral forces a suspension bridge is if it has cradled cables, and it is tacitly assumed that a suspension bridge, with the cables not cradled, is less stable in this respect. Fig. 20 shows the cross-section of a suspension bridge with cradled cables; the lines $A\,B$ and $C\,D$ give the planes of the cables, which form angles α with the vertical plane through the axis of the bridge. If the wind pushes the bridge aside until the cables are in the planes $A\,B'$, $C\,D'$, then the horizontal force which it opposes to the wind is

$$H = \frac{P}{2} \tan (\alpha + \beta) - \frac{P}{2} \tan (\alpha - \beta),$$

where P is the suspended weight of the bridge and β the angle of the planes of the cables in the new and old positions.

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If the bridge is not cradled and is moved sideways by the wind until the planes of the cables form an angle β with their original position, then the horizontal force which it opposes to the wind is $H' = P \times \tan \beta$ (see Fig. 21). Mr. Mayer.

To find what these values are in concrete cases, take the cradling to be 42.5 ft. (equal to the average cradling in the author's design); the versed sine of the cables, 400 ft.; the arc $B\,B' = D\,D' = 8$ ft.; tan α is then $\frac{42.5}{400} = 0.10625$ and $\alpha = 6^\circ\ 3'\ 54''$.

$$\beta^\circ = \frac{8 \times 180}{402.25 \times 3.14159} = 1^\circ\ 8'\ 22''$$

$$\tan \beta = 0.01989$$

$$\alpha + \beta = 7^\circ\ 12'\ 16''$$

$$\alpha - \beta = 4^\circ\ 55'\ 32''$$

$$\tan \alpha + \beta = 0.12641$$

$$\tan \alpha - \beta = 0.08617$$

$$\tan (\alpha + \beta) - \tan (\alpha - \beta) = 0.04024$$

$$2 \tan \beta = 0.03978$$

The horizontal force opposed to the wind after a lateral deflection of 8 ft. has taken place, is—

In the bridge with cradled cables $\frac{P}{2}$ 0.04024

" " parallel " $\frac{P}{2}$ 0.03978

The difference between these two amounts is $\frac{P}{2}$ 0.00046, or about 1.14% of the total amount.

It is claimed that this argument does not indicate the value of cradling for small deflections, but that it does is shown by the following computations for the two types of bridges. Taking the same general dimensions as before and assuming that the lateral deflection is only 1 ft., β will be $8'\ 33''$ and α $6^\circ\ 3'\ 54''$. $\alpha + \beta = 6^\circ\ 12'\ 27''$ and $\alpha - \beta = 5^\circ\ 55'\ 21''$. Tan $(\alpha + \beta) = 0.108766$ and tan $(\alpha - \beta) = 0.103737$. Tan $(\alpha + \beta)$ — tan $(\alpha - \beta) = 0.005029$; 2 tan $\beta = 0.004974$; tan $(\alpha + \beta)$ — tan $(\alpha - \beta)$ — 2 tan $\beta = 0.000055$. In this case the difference in resistance to wind of the two bridges is only 1.09% of the total amount, even less than it is for the larger deflection of 8 ft.

It has been stated that at the beginning of the deflection, the center of gravity of the bridge with cradled cables would rise more than that of a bridge having cables in vertical planes. The importance to be attached to this argument can be shown in the following manner: Take the general dimensions as before and let β be the angle of motion. In the bridge with cradled cables, one side rises by an amount equal to $402.25 \sin \left(\alpha + \frac{\beta}{2}\right) 2 \sin \frac{\beta}{2}$, and the other side falls by

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Mr. Mayer. an amount equal to $402.25 \sin\left(\alpha - \frac{\beta}{2}\right) 2 \sin\frac{\beta}{2}$. The rise of the center of gravity of the bridge, assuming the loads on both cables are alike, is $402.25\left(\sin\left(\alpha+\frac{\beta}{2}\right) - \sin\left(\alpha-\frac{\beta}{2}\right)\right)\sin\frac{\beta}{2}$. If the cables are not cradled, the rise of the center of gravity is $400 \times 2 \sin^2\frac{\beta}{2}$.

Since for small angles the sines can be taken equal to the arcs, the rise in the two cases is approximately $402.25\left(\frac{\beta}{2}\right)^2 \times 2$ and $400\left(\frac{\beta}{2}\right)^2 \times 2$, which shows that the difference in this respect between the two types of bridges is insignificant.

Another objection raised against vertical cables is that the resistance to wind from the inertia of the mass of the bridge will be less than when the cradling is employed. The inertia of a mass shows itself in the resistance it opposes to any change in velocity, and the force necessary to overcome the inertia is equal to the mass multiplied by the acceleration. Since there is slightly more motion, for equal lateral deflection of floor, in the upper parts of the stiffening trusses if the cables are cradled than if they are parallel, the influence of the inertia is more in the first case than in the second; but this influence acts against the wind at the beginning of the motion when the wind strains are small, and with the wind toward the end of the motion when the acceleration is negative and the wind strains are largest. The effect of inertia is therefore to increase the wind strains, and not to decrease them. The effect of inertia is, however, small while the bridge is in motion, and is nil as soon as the bridge has assumed a position of equilibrium with the wind pressure.

The effect of inertia on bridge strains, except that produced by vibrations, is apt to be exaggerated. To show this, assume six trains 1 400 ft. long to move at 60 miles an hour, or 88 ft. a second, across the author's bridge. From the moment when they cover the left half of the span to the moment when they cover the right half, a point 700 ft. to the right of the center will sink about 7 ft. For this motion it has $\frac{1400}{88} = 16$ seconds; since the velocity of this motion at the beginning and end is 0, the average velocity, $\frac{7}{16} = 0.4375$ ft., will be about half the largest velocity; this latter will therefore be 0.875 ft. This largest velocity will exist about in the middle of the motion; it is therefore acquired in 8 seconds. The average acceleration during the first part of this motion will therefore be $\frac{0.875}{8} = 0.1094$ ft. The largest acceleration will be about twice this amount, or 0.22 ft., and

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the resistance which a weight P offers to this acceleration is equal to $\frac{P}{32.2} 0.22 = 0.007\ P$. This is less than 1% of the weight of the mass considered, and can, therefore, very properly be neglected. Mr. Mayer.

Unless it is asserted that the wind will produce much larger accelerations in the mass of the bridge than would be produced by these trains, it follows that the effects of inertia can be entirely neglected in the consideration of the wind strains.

This shows that a suspension bridge offers practically the same amount of lateral resistance to wind whether the cables are cradled or not. This lateral resistance does not arise from the cradling, but from the suspension. The design becomes simpler in many respects if the cables are in vertical planes instead of cradled, and offers practically the same amount of lateral resistance to wind. The Union Bridge Company, therefore, chose the former arrangement.

In the preceding argument two cables have been assumed, but it remains valid for any number of cables, provided those on the same side of the bridge are in parallel planes. With this arrangement the strains in the cables from wind are practically nil.

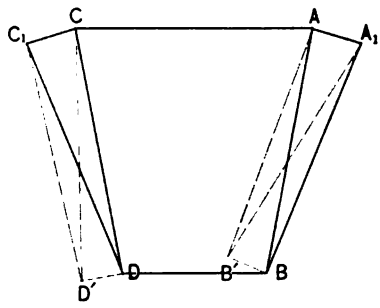

FIG. 22.

With the arrangement the author has chosen, the cables become unequally loaded when there is a lateral deflection of the bridge. If the cables are of the same length before the deflection, the outside cable to the windward and the inside cable to the leeward will be longer after the deflection than the other two (see Fig. 22). In consequence the load on the former cables will be increased, and that on the latter will be decreased the same amount. The former cables will lengthen and the latter will shorten an equal amount. The lines DD' and BB' will be arcs with centers midway between CC_1 and AA_1.

The versed sine of the cables will change by an amount approximately equal to $\frac{8 \times 14}{400} = 0.28$ ft., taking the dimensions of the author's bridge and a lateral deflection of 8 ft. in the center of bridge. This same deflection of the cables would be produced by a uniform load of 770 lbs. per lineal foot of bridge.

The horizontal force per lineal foot of bridge opposed to the wind in consequence of this unequal loading of the cables is $\frac{11}{4}{}^0 \times \frac{28}{400} \times 2 = 27$ lbs. The approximate result is therefore that an unequal loading of the cables is produced in consequence of the fact that the two cables on each side of the bridge are not in parallel planes. This un-

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Mr. Mayer. equal loading of the cables produces the same strain as a uniform vertical load of 770 lbs. per lineal foot of bridge and balances a wind pressure of only 27 lbs. per lineal foot of bridge.

The unequal cradling of the two cables on each side of the bridge produces, therefore, an increased stability, since the lateral force opposed to the wind is increased by 27 lbs. per lineal foot, but this increased stability is obtained at the cost of strains in the cables which are 28½ times as large as those which would be produced by a vertical load of 27 lbs. per lineal foot of bridge. The unequal cradling of cables is therefore an uneconomical way of resisting wind pressure and is a worse arrangement than the equal cradling of all cables.

In this respect the design in the paper is, however, a great improvement over the design submitted in the report of the Board of Engineers appointed by President Cleveland to determine the longest safe and practicable span for a Hudson River bridge at New York. That design proposed twelve cables, six on each side of the bridge, 20 ft. apart on top of the tower (see Fig. 23).

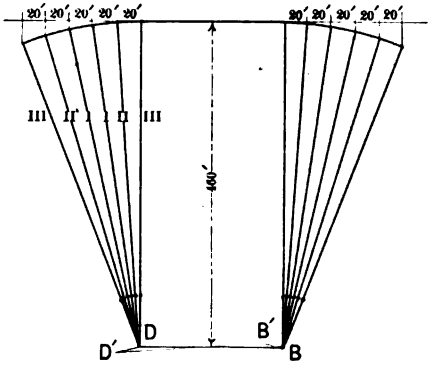

FIG. 23.

The lateral deflection $B\,B'$ and $D\,D'$ would take place on a line approximately perpendicular to the center line between the planes of the six cables. If the lateral deflection were equal to 8 ft., then the changes in the versed sines of the cables would be approximately

$$\text{I},\ \tfrac{10}{400} \times 8 \text{ ft.} = 0.174 \text{ ft.}$$
$$\text{II},\ \tfrac{20}{400} \times 8 \text{ “} = 0.522 \text{ “}$$
$$\text{III},\ \tfrac{50}{400} = 8 \text{ “} = 0.87 \text{ “}$$

The corresponding strain in the windward cable III would be about the same as that produced by a load of 2 400 lbs. per lineal foot of bridge. The wind pressure which would be balanced in consequence of this unequal loading of cables would be 61 lbs. per lineal foot of bridge. That is, 1 lb. of wind pressure balanced on account of the unequal cradling of cables would produce the same strains on some of the cables as about 40 lbs. of vertical load.

The next point in which the bridges of the author and the Union Bridge Company fundamentally differ is the design of the stiffening trusses.

The argument by which the author tries to show that it is advantageous to use a shallow stiffening truss is substantially correct in so far as it asserts that the shallower the stiffening truss, the larger a

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Mr. Mayer.

percentage of the unequally distributed load is carried by the cables without producing strains in the stiffening truss, and, therefore, the smaller the load to be distributed by the latter. It is defective in so far as it asserts that a shallow stiffening truss carrying a certain load for a given bridge is necessarily lighter than a high stiffening truss carrying a greater load for the same bridge. All depends in this case on the amount of difference in the two loads and the amount of difference in the depth of the two trusses.

There are cases where the uniform load is large in comparison with the unequally distributed load, and where the versed sine of the cables is small, in which the deformation of the cables produced by the unequally distributed load is also small; in these cases a small decrease in the depth of the stiffening truss produces a large decrease in the load it has to distribute, and it is then economical to make the stiffening truss as shallow as the allowable deformation permits.

There are other cases, especially railroad bridges, where the unequally distributed load is comparatively large, and where the use of steel towers permits a large versed sine of cables; in these the deformation of the cables produced by the unequal loading is large, the allowable deformation of the bridge produces a small diminution in the load of the stiffening truss, and necessitates a large deviation from the most economical depth; and in these a shallow stiffening truss becomes uneconomical because the loss on account of small depth is more than the gain on account of small load. The case discussed in the paper is of this class. A stiffening truss 200 ft. deep in the center, carrying nearly the whole unequally distributed load, is lighter than a stiffening truss 66 ft. 8 ins. deep, carrying 69.4% of the unequally distributed load. The stiffening truss of the author's bridge, if it were 200 feet deep throughout, would have to distribute 90.5% of the whole unequal loading. If it were lower toward the ends, and if it had a hinge at each end and in the center, it would deflect considerably more than if it were continuous and had parallel chords. Ninety per cent. is a safe estimate for the unequally distributed load which has to be carried by a stiffening truss of the shape the Union Bridge Company has adopted.

It is therefore necessary to compare a stiffening truss 200 ft. deep at the center of the half span, hinged at the center of the span and at the ends, with curved top chords, carrying 90% of the unequally distributed load, with a stiffening truss 66 ft. 8 ins. deep, continuous at the center and ends, carrying 69.4% of the unequally distributed load. The former truss has a load 30% larger than the latter.

The estimate of weight of the latter truss gives—

11 100	lbs.	per	lineal	foot	for the chords.
3 240	"	"	"	"	webs.
690	"	"	"	"	cross-braces.
750	"	"	"	"	top laterals.
15 780	lbs.	total.			

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Mr. Mayer. The section required in the truss 200 ft. deep would be one-third of the section required for truss 66 ft. 8 ins. deep, if the load were the same. The load being 30% larger, the section wanted is 43% of the section of the truss 66 ft. 8 ins. The web strains in the high truss would be 30% more than in the low one if the chords were parallel; the top chords being curved, the web strains are largely reduced at the ends.

A fair estimate gives for the high truss a saving of 54% in the weight of the chords, an addition of 60% in the weight of the webs, and an addition of 100% in the weight of the cross-bracing, or

Decrease in chords.....	5 994 lbs.	per lineal	foot of bridge.
Increase of web.....	1 944 "	"	"
Increase in cross-bracing..	690 "	"	"

This is a net saving of 3 360 lbs. per lineal foot of bridge in the stiffening trusses, or about 21.3% of the weight of the stiffening trusses.

If a hinge is made in the center there are practically no strains from changes of temperature or from deflection of cables under load. If this hinge is made so that it allows lateral motion as well as vertical motion, then the lateral motion will be so large, without being more than permissible on so long a span, that the cables take the largest part of the uniformly distributed wind pressure. How important these two advantages of a center hinge are can be seen from the fact that the strains produced in the center of the chords of the stiffening trusses by the effects of temperature changes and load changes in the length of the cables, amount to about ± 5 000 lbs. per square inch, and the strains from wind amount, according to the author, to ± 14 000 lbs. per square inch of bottom chords. These strains, together, are therefore larger than all the other strains at the centers of the chords of the stiffening trusses.

An objection has been raised to the use of a hinge in the center of the span, permitting lateral as well as vertical motion, that it would be difficult to make such a detail. If the lateral deflection is so large that the cables balance the wind pressure near the center of the span, then this hinge will have to resist no tension or compression but only shear, and there are several methods of constructing it. One of them is connecting the ends of the top chords of the first half span with the ends of the bottom chords of the second half span, and the ends of the top chords of the second half span with the ends of the bottom chords of the first half span, by means of vertical wire ropes which will transfer shear in both directions, and will allow considerable lateral deflection, especially if they have considerable length, without producing large tensions or compressions in the chords of the stiffening trusses. The same connection might be made by means of flexible plates instead of wire ropes.

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In the preceding comparison of the weight of two stiffening trusses, one 66 ft 8 ins. high and continuous, the other 200 ft. high and hinged, no consideration has been taken of the very large strains in the chords of stiffening trusses, which are the consequence of the absence of hinges, and would, if considered, make the comparison still much more favorable to the hinged truss. The other differences between the strains of a hinged truss and those of a continuous truss have also been neglected through the whole argument. These differences are not very large, and are, on the whole, in favor of the hinged truss. Mr. Mayer.

In the argument showing that the stiffening truss has to distribute only part of the moving load, in the author's bridge, 69.4% of it, it is assumed that the other 30.6% of the moving load will produce at the lower ends of the suspenders exactly the same deflections as the 69.4% of it produce at the corresponding points of the stiffening trusses. This assumption is based on the single fact that the deflections produced at one-quarter of the span by the two loads before mentioned at the ends of the suspenders and in the stiffening truss are alike. Since the deflections produced in the cables and stiffening trusses by any loads are dependent on different and independent variables, this assumption cannot be absolutely and generally true. This vitiates to a large degree the calculation of a shallow stiffening truss and to a much smaller degree that of a deep one. A considerable saving, which has not been considered, can be made by varying the sections throughout the length of the stiffening trusses, but this is only advisable where it is possible to make accurate strain sheets, as is the case in a hinged truss.

The foregoing reasons induced the Union Bridge Company to choose, in its design for a suspension bridge across the Hudson for six railroad tracks, cables in vertical planes and hinged stiffening trusses of 200 ft. depth.

The difficulty in the author's design for the cables, which arises from the impossibility of making sockets as strong as the ropes, might be overcome by bending the latter about a half-round saddle on top of the towers before inserting them into the sockets. In this case the maximum strains in the ropes at the sockets would probably be so much smaller than near the saddles that the sockets would not be the weakest parts of the cables. Certainty on this matter could be obtained easily by tests.

F. Collingwood, M. Am. Soc. C. E.—The author states that the strands of the cables will be adjusted under tension at the works, so that no further adjustment will be required in the field. The work of putting the ropes in place is described, and apparently this is to consist of simply carrying them across the span one by one, and placing the sockets in the places provided for them. In this connection the speaker asked whether consideration has been given to the change in Mr. Collingwood.

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Mr. Collingwood. elevation which will be caused by the change in length, resulting from drawing in the ropes to form the cables. The diameter at the points of attachment scales in the cuts in the neighborhood of 7½ ft., and allowance must be made in the length of each strand for the position it finally occupies.

Again, it is problematical whether it is practicable to bring numerous ropes hanging freely in space into a compact cylindrical form without building up a core first, and possibly adding successive layers. The lateral movement of any one of the many ropes will interfere with the final result, and such movement is almost certain to be caused by the operation of compression. The fact, also, of the wires in the ropes twisting spirally around the exterior will interfere with the free movement which straight wires would have.

In laying up the cables of the existing East River Bridge, the strands were each seized at intervals of 2 ft. 4 ins. throughout their lengths. After the first twelve strands in each cable were completed and in place, the central seven were made into a core. In each case the previous seizings were removed sufficiently far in advance to allow the wires to be spread out into their true position, which was done by means of mallets and pressure. Finally the nineteen strands were brought into a compact whole.

The exact length of every strand when in position in the cable was calculated as nearly as possible, care being taken not to make it too short. When the strand had been lowered to place, its length was finally adjusted by the position it occupied vertically at the center of the span, those strands which were to be drawn in the most hanging the lowest, the central strand being the guide.

The work of adjustment is undoubtedly increased by cutting the ropes at the towers and the increased weight of material due to it is a large item; the question naturally arises as to whether it is a necessity.

Mr. Cooper. THEODORE COOPER, M. Am. Soc. C. E.—In discussing this thoughtful paper, it should be borne in mind that the number of tracks, character of the train service and local conditions will vary for different localities, and that even at any particular locality there may be a wide difference in the views held as to the capacity that should be given to such a structure.

The only true test of the relative merits of different systems of bridges to accomplish best the desired result is a comparison of costs for structures under a uniform set of requirements.

The author has adopted a cable system formed of coiled-wire ropes, a stiffening truss continuous over the piers, and a maximum allowed deflection at the quarter-span, as essential features of his study. Whether this combination will give an economical and satisfactory structure is at least questionable.

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Mr. Cooper.

Coiled-wire ropes have been used for suspension bridges, and have attractive features from the point of view of more rapid erection of the cables. The longest span formed of coiled-wire ropes known to the speaker is the Middle Harbor Bridge, at North Sydney. It is 1 026 ft. long, and has, judging from a photograph, a center-hinged stiffening truss; it is not a railroad bridge. Cables of this kind have been advocated by General E. W. Serrell for bridges over the Hudson River at Fort Montgomery and at New York City.

Coiled-wire ropes must have a lower modulus of elasticity than cables formed of straight wires. While the elongation which measures the modulus is in the straight wire an actual elongation of the metal, in a coiled rope it is only partially an elongation of the metal, the rest being due to the elongation of the coils of the individual wires forming the rope. The elongation of the coils will depend, to a more or less extent, upon the rigidity of the core and degree of hardness with which the coils are wound, for the only thing which prevents the straightening out of the coils is the radial resistance to compression of the core and inner strands.

The modulus of a coiled-wire rope is partly tensile elongation of the metal, partly torsion, and partly elongation of the coils.

It is natural, therefore, to expect coiled ropes to have a considerably lower modulus than straight wires, and also that the moduli would be variable for different ropes or parts of the same rope.

As far as the speaker has been able to discover, actual tests confirm the above theoretic expectations. Tests submitted to him by General Serrel, with his proposed plan for the North River Bridge, which were made at the Watertown Arsenal, showed that the moduli of different ropes for different loads varied as widely as 12 500 000 to 24 000 000 lbs. Between the loads of 30 000 to 60 000 lbs. per square inch a hawser-laid rope gave a modulus of 16 500 000 lbs., and ropes formed somewhat similar to those proposed by the author gave moduli from 21 500 000 lbs. to 24 000 000 lbs.

The abstract of tests given by the author shows that under different degrees of loading the elongations of the coiled-wire ropes exceeded by 30% to 60% the elongations of the straight ropes. The coiled ropes, however, after being under strain for some hours, gave a fair modulus of about 25 000 000 lbs.

The fuller reports* are very interesting, and make a better showing between the coiled and straight-wire ropes than the abstracts did. Test No. 8 271 on straight-wire rope of plow steel, and test No. 8 275 on coiled-wire rope of plow steel, may be taken as typical of the general results. The elongations for each 10 000 lbs. in percentages are tabulated on the next two pages.

* Sent by the author to the writer after the meeting. A copy of the complete report of the tests is filed in the Library of the Society.

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Mr. Cooper.

No. 8 271.—Straight-Wire Rope of Plow Steel.

Strain per square inch. Pounds.	Elongation for 10 000 lbs. Percentage.		
10 000	0.000		
20 000	0.0632		
30 000	0.0432		
40 000	0.0386		
50 000	0.0410		
20 000	0.0517		
30 000	0.0473		
40 000	0.0428		
50 000	0.0416		
60 000	0.0406		
20 000	0.0416		
70 000	0.0447		
20 000	0.0483		
80 000	0.0428		
70 000	0.0359		
60 000	0.0364		
70 000	0.0361	Average	0.0362
80 000	0.0364		
90 000	0.0398		
80 000	0.0352		
70 000	0.0363	Average	0.0361
60 000	0.0368		
20 000	0.0580		
60 000	0.0447		
70 000	0.0372		
80 000	0.0364	Average	0.0370
90 000	0.0374		
etc.			

Ultimate strength of rope..... 188 980 lbs.
" " " wires ..... 226 300 "

The negative set shown on relaxing the strain and the difference in ultimate strength between the rope and the wires from which it was formed indicate that the wires of the rope were not pulling equally. The elongations, therefore, are greater, and the resulting modulus less, than would be the case were all the wires equally strained. For long ropes this failure to pull equally would be less than for short samples suitable for use in the testing machine. The small differences in lengths of the wires and also any slipping of the wires in the sockets would be relatively less important, as the ropes increased in length.

It will be seen that when this rope had been strained to 80 000 lbs. and then tested back and forth from 90 000 to 60 000 lbs., the elonga-

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tion became quite uniform at about 0.0361, giving a modulus of about 27 700 000 lbs. Perhaps this modulus would have been higher if this rope had been treated similarly to the coiled ropes 8 273 and 8 275, viz., kept under a strain of 60 000 lbs. for a number of hours and then tested back and forth between limits not exceeding 10 000 lbs. It would be very interesting to have had exactly similar tests upon both kinds of ropes and single wires, that their moduli could be compared. Mr. Cooper.

No. 8 275.—Coiled-Wire Rope of Plow Steel.

Strain per square inch. Pounds.	Elongation for 10 000 lbs. Percentage.	
10 000	0.000	
20 000	0.0520*	
30 000	0.0513*	
40 000	0.0527*	
50 000	0.0510*	
60 000	0.0527*	
50 000	0.0415	
60 000	0.0420	
50 000	0.0415	
60 000	0.0415	
50 000	0.0412	Average.....0.0416
60 000	0.0417	
50 000	0.0416	
60 000	0.0416	
50 000	0.0400	
60 000	0.0400	
50 000	0.0400	
60 000	0.0401	Average.....0.0400
50 000	0.0401	
60 000	0.0400	
70 000	0.0463*	
60 000	0.0403	
70 000	0.0408	Average.....0.0404
80 000	0.0517*	
70 000	0.0396	
80 000	0.0408	Average.....0.0402
90 000	0.0581*	
80 000	0.0397	
90 000	0.0415	Average.....0.0406
etc.	etc.	
Ultimate strength of rope.....		177 690
" " wires.....		226 300

* Held at 60 000 lbs. for 16½ hours.

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Mr. Cooper. It will be noticed that when a new strain greater than a previous strain is put upon the rope (as marked by*), the elongation is large; but when the strains are not varied more than 10 000 lbs. back and forth, the elongations are smaller and more uniform. The wires probably move slightly as long as the strains are increasing, making the greater elongations; but after they have taken on the new position, motion does not take place for limited strains of a less intensity. If the rope is held for hours under one strain, the wires complete their movement and then the elongation becomes the true elongation—free from any movement of the wires on themselves.

The wires in the coiled rope appear to have pulled together—as would be expected from their twisted condition—better than in the straight rope.

The modulus of 25 000 000 lbs. as compared to the modulus 27 700 000 lbs. of the straight rope is a favorable result, but in longer ropes or bridge cables it is probable that the coiled ropes would not show much better results, while the straight ropes should be somewhat better. The ultimates would probably be better in an actual cable, but the straight ropes would get the greater increase. Neither, however, could be expected to give as good an ultimate strength or modulus as would be due to straight wires. Judging from the ultimates of the straight ropes and the single wires, the modulus of the wires should have been 30 000 000 lbs. or more.

The author has selected a stiffening truss continuous over four supports, which will be about 25% stiffer than a truss supported at the ends. As the cables will be more elastic than straight-wire cables, the combination of a stiffer form of truss with the more elastic form of cable will not be in the line of economy, for it will increase the proportion of the load which must be carried by the truss, and also increase the strains due to the rise and fall of the cables through temperature changes. Whether the loss of economy will be compensated for by the absence of a central hinge and end fastenings is questionable.

The author's description of the functions of the stiffening truss for different classes of bridges does not cover the whole case. The speed of the moving load is also an important factor. A cable that might be sufficiently flexible to accommodate itself to slowly moving loads without injury would be seriously affected by the same loads moving at far higher speeds.

The author has selected as a permissible deflection at the quarter span a depression of 3½ ft., with its accompanying rise at the opposite quarter, for the purpose of proportioning his stiffening truss. That such a distortion of the cables would not in itself be harmful is true; but when such distortions, if they occur, must be reversed at opposite quarter spans in the space of a few seconds, under trains moving at 30

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to 40 miles per hour, they become a serious matter. On the only rail- Mr. Cooper.
road suspension bridge in use for heavy trains, the Niagara Suspension Bridge, trains are not only limited in weight, but also to very slow speeds, 4 to 5 miles per hour. Considering the limitation that this restriction puts upon the capacity of this bridge, it is fair to assume that they have not found high speeds favorable.

It is not of course claimed that there will be undulations traveling from one end of the cables to the other of any such magnitude as 3½ ft., as the inertia of the mass would not permit it; but the energy corresponding to such an undulation must be destroyed somehow. That this will not produce dangerous and destructive results in the cables or other parts of the structure should be well established before being adopted as a feature of the design.

That it would tax heavily the suspender system seems certain. For a railroad suspension bridge, reliance should not be placed on the frictional resistance of cable clamps for suspender loads. While it may be possible to obtain sufficient friction to resist the slipping of the clamps, the compression put upon the cables to obtain such friction may be very injurious to the cables. Moreover, any slipping of the clamps injures the cable wrapping. It is stated that some of the clamps of the Brooklyn Bridge slipped in the early days of its construction.

For long spans like the one under consideration, the greater versed sine, obtained by use of metal towers, makes the angle of the cables much steeper than those of the Brooklyn Bridge (about 50 per cent.).

In addition, the heavier train loads make it more important that the connection of the suspenders to the cables be positive.

In the specifications for the proposed North River Bridge, the speaker required that "the pull of the suspenders in the direction of the cables must be taken up otherwise than by the friction of the cable bands produced by bolts through their flanges. Only 20% of the normal pressure upon the cables, produced by the load on the suspenders, shall be considered as resisting this pull. The remainder must be taken up by supplementary cables or ropes running from the saddles to the suspender connections or some equally acceptable device."

Fig. 24 shows the general method that was intended by this clause. The inner sleeve is clamped to the cable as a bearing for the free saddle carrying the suspenders. The saddle is held by the supplementary ropes extending from one saddle to the other, and anchored at the towers. By this means the saddle can accommodate itself to any sudden or undue loading without the chance of causing slipping of the cable clamp out of position.

If the vertical load on one suspender is called V, the pressure in

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Mr. Cooper. a normal direction at the suspender joint at the highest point (for the plan under discussion) will be $0.903\,V$, and the tangential pull $0.425\,V$. If 20% of the normal pressure is assumed to resist the tangential pull, the suspender load on the highest suspender only needs a tangential resistance of $0.244\,V$. Summing up all the tangential pulls for the whole series of suspenders, it will be found that, when all the suspenders are fully loaded, the maximum tangential pull at the towers will be about $3\,V$, or the supplementary ropes at the towers need be only sufficient to resist the load on three suspenders, or 100 ft. of

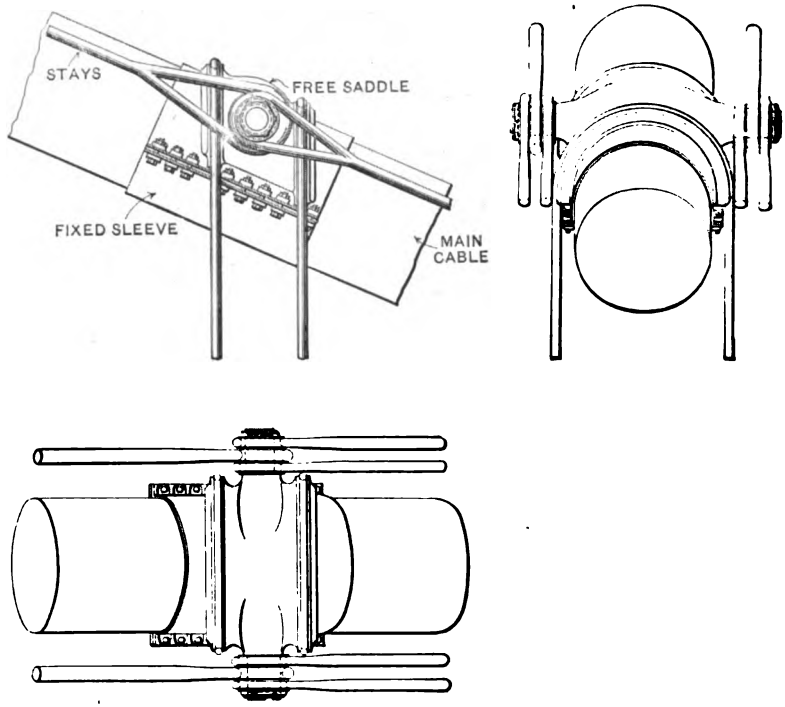

FIG. 24.

bridge, the suspenders being $33\frac{1}{3}$ ft. apart in this plan. As these ropes can be loaded safely with higher strains than the suspenders, if they are loaded the same as the main cables they will only equal $1\frac{1}{2}$ times one suspender, or $4\frac{1}{2}$ sq. ins., divided into two ropes, one on each side of the main cable. As these $4\frac{1}{2}$ sq. ins. will carry loads that otherwise would have been carried by the main cables, $4\frac{1}{2}$ sq. ins. can be deducted from the main cable areas for their whole length between towers, so that, instead of being an additional cost, they will produce a saving.

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E. GYBBON SPILSBURY, M. Am. Soc. C. E.—Mr. Cooper calls attention to the fact that the results of the experiments on wire ropes mentioned in the paper do not correspond with those found by Gen. Serrell some years ago. The discrepancies between the results appear to be due to the difference of the structure of the ropes or cables tested, and also to the difference of the methods used in socketing. Gen. Serrell made his tests on a rope which he describes as hawser-laid. This rope, as generally known to the trade, is made up of six strands, each consisting of twelve wires twisted round a hemp center, these strands being again laid together round a hemp center. With a cable made in this manner, it is hard to see how any one could expect to obtain any regular or determined modulus. Even if the term "hawser-laid," used by Gen. Serrell, was a mistake, and he meant an ordinary six-strand cable with wire instead of hemp centers, the results would have been the same, as the tensions of the wires in each strand would be different from the tensions of the strands in the rope, and it would be a practical impossibility to bring them into unison. The twisted cables tested by the author were in reality simple strands, and consequently not subject to the same objections. In one set of these test ropes the wires were all twisted with a long lay in the same direction; the other set had each layer of wires coiled in opposite directions. The advantage of this latter construction is to balance the internal strains and obviate the natural tendency of coiled ropes to revolve on their central axis when under strain. Whether this advantage will be found to overbalance the possibility of internal shearing strains being set up in the cable, due to the crossing of the wires, as suggested by Mr. Cooper, the speaker was not prepared to say, nor did it seem material to the general question, as the results for modulus were practically the same in both cases. Mr. Spilsbury.

Admitting the feasibility of using coiled strands in such a bridge construction, the success of the socketing of these strands is of the highest importance. The author calls attention to the fact that this is possibly the weakest point of the structure, and bases this opinion on the result of the tests before alluded to. As a fact, the first breaking of the cables under test always occurred on the outside layer of wires, but inside the socket. It was evident from this fact either that the inside shape of the socket was defective, as suggested by the author, or that a certain amount of slippage took place in the inner layers of the rope, which, not being participated in by the wires of the outside layer, resulted in imposing an excess of strain on these latter, and their consequent breakage.

The sockets used were of the ordinary type for this kind of rope. They were cylinders of soft machinery steel bored out conically for the spread of the wires of the strand. The method of socketing was as follows. The end of the rope being passed through the socket, the outside

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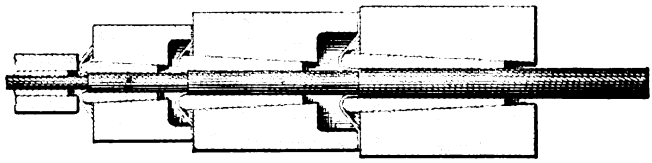
Mr. Spilsbury. layer of wires was partially untwisted, and each wire separated and spread out against the taper surface of the socket. A split wedge-shaped thimble, having the same taper as the socket, is then put around the inner coil and driven home. In the interstices between each wire are driven taper wedges. The first layer secured, the wires of the succeeding layers are similarly spread out, and all the interstices filled in with similar wedges and thimbles, all being finally forced down into place, either with a screw or hydraulic press. Theoretically, if the result of this work was perfect, the central wires, being the shortest, should have broken first; whereas, as stated by the author, the outside wires were the ones which failed. The author attributes the failure at this point to be due to the ridge or shoulder which existed in each coupling at the point where the taper merged into the straight opening. The speaker's impression is that this ridge cannot account alone, if at all, for the breakage, owing to the fact that the material composing the socket was soft steel, while the wires of the rope were excessively hard. This was shown by cutting open one of the sockets after the test, when it was found that all of the outside wires were imbedded to about one-third of their diameter in the steel of the socket, which, under the pressure, had flowed up between the wires, and in some



instances been forced up even between the wedges in a thin film. It is his opinion that the failure of the sockets used to develop the full tensile strength of the rope was due to the difference of the coefficient of friction between the wire and the metal of the socket, in the first place, and the wires among themselves, in the second place, thus allowing these latter to slip on themselves and impose excessive strain on the outside wires, which, under the increased frictional pressure, were unable to give correspondingly. In order to avoid these defects, a new socket has been designed, specially adapted for smooth-coil cables. This socket (Fig. 25), which has been named, from its shape, the steeple socket, is really a combination of several sockets one back of the other, and each one having to take care of only a certain number of layers of wire. The cable being passed through the first section of the coupling, the outside layer of wires is spread out to conform with the taper in the socket, while the rest of the rope remains intact and goes through to the second socket. Taper wedges and thimbles are driven in between these outer wires and the central core, and smaller wedges are driven down between each of the outer wires, thus insuring an even pressure on each of them. The ends of these wires being then bent

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over the outside of the socket, the second socket is fitted down on them, and the second and third layers of rope are spread out in the taper of this socket, and secured in the same manner as the outside wires were in the first one. This is repeated in each following socket, until every wire in the rope has been individually secured. While the length of such a fastening is somewhat greater than the ordinary socket, it is not so much so as would be imagined, as the size of each following socket is, of course, much smaller than the preceding one, it having less work to perform. Several of these fittings have been tested on different sizes of ropes; the rope has broken invariably entirely outside of the sockets, thus showing that the fastenings were always stronger than the cable. Mr. Spilsbury.

W. H. BREITHAUPT, M. Am. Soc. C. E.—The cutting of the cables at the towers in the author's design involves a great deal of work and additional weight. To offset this it is held that a stiffer and stronger wire can be used in the cables on account of avoiding its weakening by the abrupt bend at the towers, that erection will be much simplified, and that the backstays can better be given any desired direction. Mr. Breithaupt.

The weakest parts of the ropes forming the cable, as is brought out in the paper, are the sockets. The vertical angles between the backstay and main cable ends are 129° 17′. Subtending this with a chord of 12 ft., which would give a large enough saddle if the cables are fixed to the towers, will give a tangent-to-cable circular arc, the radius of which is 13.99 ft. To bend a 2⅛-in. wire rope, even if of the stiffest wire used for such rope, over a cylinder of 28 ft. diameter will impair its strength very little; the sockets would still remain considerably the weakest part of the rope. The bending over this cylinder of 28 ft. diameter would not be of the 3-ft. cable, but only of the 2⅛-in. wire rope, as is obvious from the method of construction.

The three shorter ropes proposed, the heaviest weighing 33 420 lbs., could be more easily transported than a rope in one piece, of 56 980 lbs., reaching from anchorage to anchorage. Transportation of the latter would not be impracticable; if found preferable a thinner rope coming within the desirable weight limit for transportation could be used. The work of paying out one rope from anchorage to anchorage should not be more than that of paying out the three proposed sections, and would be done from a better working base; though it must be borne in mind that the three shorter sections might be reeled off directly from the barges on which they were transported, located at the tower bases.

Friction and the angle of the cable over the tower would in themselves suffice to transmit horizontal stresses from cable to tower, and fastening could easily be further safeguarded by clamps. The proper pulling of the backstay and the main part of the cable against each other would be a matter of adjustment, in position and inclination, of

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Mr. Breithaupt.

the saddles, and there would remain the same freedom as by the proposed method of giving the backstays the most desirable direction.

There is another objection to the proposed method of fastening the cables to the towers. The cables are at this point increased from a diameter of 3 ft. to one of about 6 ft. The center rise and fall of the cable from stresses and the assumed temperature changes is 10.26 ft. between extremes. This causes material detrimental bending of the cable at the tower fastening, even on a 3-ft. diameter of cable, and is proportionally worse with a 6-ft. diameter.

Making the cables continuous from anchorage to anchorage and using eight saddles, instead of cutting and using the proposed connections, gives an approximate difference in parts, weight and cost, as follows:

4 048 sockets.....	145 728 lbs. at 10 cents	$14 572 80
Details at tops of towers 7 271 376		
Less eight saddles and connections, say..... 600 000		
	6 671 376 lbs. at 5 cents =	$333 568 80
		$348 141 60

The proposed towers do not require longitudinal stiffness, and would be lighter with a narrower longitudinal base, the material saving being in the bracing. In that case the weight would, however, have to be spread again to cover the required foundation area. A continuation of the idea of fastening cables to towers would suggest using rocker bents for the towers. Such bents could be proportioned for direct pressure only, and be much lighter than towers of the ordinary form. They would not be practicable for towers of the proposed magnitude; but for shorter spans, with towers of not over 175 ft., they would afford very considerable advantages.

On the whole for towers approaching the proposed magnitude the advantage remains with the customary arrangement of movable saddles if the towers are given a wide enough base longitudinally to ensure their stiffness in that direction.

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CORRESPONDENCE.

GUSTAV LINDENTHAL, M. Am. Soc. C. E.—Although the suspension type was used for the earliest iron bridges, it was the latest for which correct theories of calculation as regards strains from moving loads were developed. The dimensioning of the cables, towers and anchorages was always a simple matter, and, to judge from the records of older suspension bridges, was correctly done in the little light the early engineers had on the strength of iron. Before the era of railroad building, heavy concentrated loads did not need to be considered, being easily avoidable by street regulations. Mr. Lindenthal.

The suspension type for railroad loads was a different problem. Only two suspension bridges for railroads were ever built; one on the braced double-chain system by Engineer Schnirch over the Danube Canal in Vienna in 1860, replaced in 1886 by an iron arch bridge; and the other, the famous wire-cable bridge over the Niagara River, by John A. Roebling, in 1854, being replaced now, also, with an arch bridge.

At the time of their construction no reliable theory of the different systems of stiffening existed, and it is not surprising that both bridges proved inadequate for the concentrated loads of trains, which are growing heavier every year. In spite of the insufficient stiffening, both bridges showed up remarkably well, and together with the Brooklyn suspension bridge, which, although not adapted for railroads, is nevertheless subject to the concentrated load of 100-ton passenger trains meeting on two tracks, augured favorably for the suitability of the suspension type for long railroad spans, when properly designed in the light of modern theory.

As the paper deals with a span corresponding to the dimensions required for a railroad bridge over the North River, for which the writer made the first studies and plans, beginning some twelve years ago, and when a span of 3 000 ft. was looked upon as impracticable, even by some bridge engineers, he read with a special interest the author's views on the subject.

The paper treats of a stiffened suspension bridge in its simplest form; that is, of a combination of cables and stiffening trusses for the middle span, with the backstays of the shore spans carrying only their own weight. The main span is affected only by the loads occurring on it, and the calculations are thus not complicated by loads suspended from the backstays. The bridge would be sufficient for six railroad tracks.

The author invites special attention to two features. One is the continuous stiffening trusses extending beyond the towers, the other his method of cable construction.

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Mr. Lindenthal.

The first is the most commendable feature of the design. It would look better, however, if the trusses were extended to the anchorages. The possible turning of the tracks into a curve on the New York side before they reach the towers may explain the unsightly arrangement of one end of the design, but there appears no need for repeating it at the other end of the bridge for the sake of symmetry.

The special merit of the continuous stiffening girder consists in the shifting of the points of contra-flexure under the action of the moving load. It is a prevention of and a protection against localized bending strains in the cables, and a great advantage over the center-hinged girder, which exposes the cable to sharp and continuous bending strains at the center.

The continuity of the girders through the towers is also a good feature, contributive to greater stiffness. It protects the cables near the towers against the effects of sudden loading from fast trains better than girders not continuous through the towers.

Nothing of any account would be gained, theoretically or practically, in varying the sections of the chords or of the web members for such a girder as in the design, and the author chose wisely in adopting a uniform section for the suspended part throughout.

There is no single quantitative law for the bending and shearing strains in continuous stiffening trusses. For such having the heaviest chord sections or greatest height, or both, at the middle, the deflections will be different under the same moving loads as for trusses having them at the quarters. For different assumptions of height or sections for the trusses, there will result differently distributed deflections. While the preliminary calculations for working out, or balancing, the design in the rough, as it were, are simple enough, the final computations, even with the facility of uniform moments of flexure, are sufficiently involved to make the complication from unnecessary variations of section and height undesirable. They offer no compensating advantages.

The writer agrees with the author that the middle hinge in a stiffening truss does not have all the good points claimed for it. The simpler statical calculation for this kind of truss is no advantage, since equal reliability is possible for the more intricate calculations of the continuous girder; they concern the engineer alone, and cannot be counted an objection. The necessity of carrying the top wind bracing down to the lower wind truss at the middle hinge is a grave objection; and as regards the supposed freedom from temperature strains, it is a myth. Theoretically, as well as practically, the hinged stiffening girder requires more material than the continuous girder for equal unit strains. Making the girders high would in both systems lighten the chords and show an apparent saving, were it possible to ignore the temperature effects. They can, however, be no

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more ignored in hinged than in continuous girders. For equal height of trusses, the moment areas from temperature strains are the same in both systems, although differently distributed. Prof. Melan, of Austria, a leading mathematical authority on suspension bridges, and an expert engineer whose judgment the writer considers of weight, expressed himself likewise in favor of the girder without middle hinge in a conversation on this subject. For an important city bridge, the ungainly appearance, particularly of high center-hinged girders, should alone be a decisive objection, and it is merely a question of time when this kind of girder will be classed with the Bollman, Post and other obsolete forms of trusses, which likewise were once thought to have special merits of their own. Mr. Lindenthal.

The so-called cantilever feature of the stiffening girder produces a considerable saving in weight as compared with the ordinary continuous girder. An approximate calculation shows that for the latter the excess load absorbed by the cable would be only 7% instead of 9.4%, as in the author's design. The simple continuous girder would need to be 22% higher and 8% heavier between towers, including due allowance for the chords tapering from the quarters towards the towers. The cables would be heavier by 3%, and the weight per lineal foot would be 40 600 lbs. instead of 39 000 lbs. The cost of the metallic part of the bridge alone would be increased by $300 000 over the author's design, which in this feature, therefore, possesses also the merit of economy over the simple continuous girder, and still more so over the center-hinged girder.

It is not quite clear why the ratio of dip (using this word for the deflection or versed sine, improperly so called, of the cable) to span was chosen as one to eight, possibly for the sake of high, flexible towers. Flexibility is sometimes an advantage, but great metallic height is a disadvantage. The larger amount of elastic contraction in the towers from the pressure of moving loads, together with the greater sagging and stretch in the longer backstays, causes the tower tops to swing out towards the main span more than with shorter towers; the greater height is also more expensive. It is doubtful whether an economical advantage is gained over a flatter catenary, which would look better.

Flatter catenaries have greater initial rigidity, as the writer would term it, namely, a larger proportion of live load is required to distort the cables. The effect of it, like that of cradled cables on lateral stability, is much more noticeable and beneficial in practice than mere theoretical considerations would indicate.

The excess live load on the loaded half, deflecting 3.5 ft. at the quarter, would increase from 9.4% of the dead load (with the first suspender 200 ft. from the tower) to 11.3% for a dip of one-tenth the span, and to 13.5% for a dip of one-twelfth the span. Thus, assuming for

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Mr. Lindenthal.

easier comparison the dead load (39 000 lbs. per lineal foot) carried by the cables to remain the same for different ratios of dip to span, the excess load, which in the design takes 3 675 lbs. out of 12 000 lbs. and represents 30.6% of the live load, would take 4 400 lbs., or 36.7% of the live load for a dip of one-tenth the span, and for a dip of one-twelfth it would take 5 260 lbs., or 43.8% of the live load, all for the same deflection of 3.5 ft. at the quarter point. The stiffening trusses would therefore become much lighter, although the cables would become heavier.

The length of span itself has no influence on the percentage of excess load as expressed per lineal foot, *i. e.*, if the span were 1 000 ft. instead of 3 200 ft. (or, virtually, 2 800 ft., as in the author's design), the percentage of excess load would be the same, provided the dead load, the dip of the cable and the deflection at the quarter were the same; but when the length of span is given, the proportion of live load for which the stiffening girder must be dimensioned will be smaller as the cable curve gets flatter, and as the deflection in the quarter is allowed to be greater.

There will be, then, for a given span and live load, and for a given deflection at the quarters, some combination of dip of cable and height of stiffening truss which will result in a design requiring the least amount of metal for the superstructure and towers. Formulas deduced for that purpose are, however, of little practical value, since with the different prices for cables, towers, stiffening trusses, floor construction and masonry and with different details, it is not always the lightest structure which is the most economical. In any given case the best combination will be obtained by the preliminary calculations of a few test designs, and it will then be found somewhat a matter of give and take as regards the different elements affecting the design.

Inasmuch as small variations in the deflection at the quarter will produce very sensible differences in the weight of the stiffening trusses, cables and towers, it should not be made smaller than necessary. In ordinary railroad bridges a deflection equal to 0.001 of the span is considered no impediment to the fastest trains. Indeed, in most of the cantilever bridges built, the deflections are much greater. The fastest trains are some passenger and express trains, hardly exceeding 1 500 lbs. per lineal foot and 600 ft. in length. Assuming such trains on six tracks to meet on the half span of 1 400 ft., there would be 2 700 tons more or less distributed. It is all sufficient to assume that the six passenger trains, running at 50 miles per hour, should cause no greater deflection than 0.001 of the half span, equal to 1.4 ft. The steepest change of grade would be 0.04% for a length of 200 ft. On this assumption, the 8 400 tons maximum of the author would produce a deflection of 4.35 ft. The change of grade would be 1.24% for a short distance, an entirely permissible grade. The load of 8 400 tons

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may never meet upon the half span, as it would be an extreme case, while six fast and heavy passenger trains may not meet once in a year with the densest traffic. Mr. Lindenthal.

As mentioned by the author, the dead load could be reduced if the allowable deflection at the quarter were increased. An increase from 3.5 to 4.35 ft. would require an excess load of 11.2 instead of 9.4 per cent. The stiffening girder would be lighter, and slightly less in height for the same unit stresses. The dead load would be reduced from 39 000 lbs. to 37 800 lbs., with all the corresponding saving in cables, towers, etc., from the smaller dead load; that is, for the same dip of cable as in the author's design, one-eighth the span.

If a combination of stiffening truss with a flatter catenary would cost no more, the latter would be preferable. The saving of steel in the towers, for the same dead load of the main span, would be considerable. It would amount to 80 ft. in height for a dip of one-tenth the span, equal to about 7 840 000 lbs. and to 133 ft. in height, or to 3 000 000 lbs. for a dip of one-twelfth the span. Against this saving would be, as a set-off, the increase in the weight of the backstays and in the size of anchorage.

The writer is not prepared to say, without a more extensive investigation than a general discussion would warrant, whether the saving in the weight of the towers and stiffening trusses at 4 cents per pound would balance the increase in weight in the cables and backstays at 7 cents per pound, plus the greater expense of the anchorages. This much can be said, however, that in all cases where the material in the cables is as cheap or cheaper per pound than that in the stiffening trusses and towers, as would happen with eye-bars suitable for shorter spans, there will be a decided saving in cost for a flatter catenary. Almost all the older suspension bridges were built with flat catenaries of one-twelfth to one-fifteenth the span, and to this condition their comparative stiffness in the absence of any adequate stiffening device must be ascribed. It was the result of sound engineering intuition, as even Prof. Rankine's theory, the first one published, required the same weight and strength of stiffening truss for steep and for flat catenaries alike.

The larger temperature strains in stiffening trusses caused by flatter catenaries are not a serious matter, if the former be not made unnecessarily high.

Trusses of hard steel will need to be higher than the equivalent trusses of soft steel for same deflections. The height should, however, not be greater for continuous girders than one twenty-seventh of the span, because in higher girders the influence of the web upon the deflections could no longer be neglected. The short reaches between points of contra-flexure for certain positions of the moving load would, in higher trusses, offer less than the theoretical moment of resistance,

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Mr. Lindenthal.

measured by the chords alone, and the actual deflections would be greater than the calculated ones. A very long or a very heavy span can have low stiffening trusses for the reasons stated, but with the span or dead load becoming smaller, the stiffening trusses will, for the same degree of stiffness, have to be made higher proportionately to the span.

The height chosen by the author in the provisional manner he describes is, therefore, a proper height, and another good feature of his stiffening truss is the riveted connections, with web members riveted up also at their intersections. It greatly helps to diffuse the bending strains throughout the frame, and the so-called secondary strains at all the connections and intersections are a decided gain on the side not only of greater stiffness, but of greater safety, contrary to the received theoretical considerations on this subject.

It is not necessary to combine the bending strains from live load with those from wind pressure and temperature for proportioning the chords, as advised by some engineers, because all the maximum strains can never occur together in such a large structure.

No trains will crowd upon the bridge during a gale strong enough to upset the cars. It will always be prudent to combine the different maxima for aiding the judgment, and keeping on the safe side, as the author has done; but no unbending rule for dimensioning would be justified for conditions of extremely rare occurrence.

In view of the rules prevailing for the dimensioning of bridge members subject to alternate tension and compression, the author seems to consider 20 000 lbs. per square inch a large concession, equal to 40 000 lbs. between the extremes of positive and negative, in 60 000 to 65 000-lb. steel. The writer is of the opinion that for the very rare maxima, the limit can be made higher with this steel, and still higher with hard steel. The usual formulas for alternating strains are precautionary rather than correct, and, in the absence of more accurate knowledge, perhaps justified for short spans and fast trains, but in long spans, where the reversals take place slowly, they are wasteful. It should make a difference whether the alternations take place at the rate of six per second or six times a century; whether they are in the form of shock, or in the nature of gradual or cumulative strains. There is, as yet, not sufficient light on that question.

As regards the suitability of hard steel for members subject to alternate tension and compression, the conclusions of Sir Benjamin Baker are perhaps the latest on this subject. He made tests for the Forth Bridge with spindles weighted at the ends, from which he believed that although hard steel endures repeated bending under given absolute stress better than wrought iron and soft steel, yet it excels them much less in alternating tensile and compressive strength and elastic limit. This, to the writer's mind, is a conclusion not fully

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justified by his tests. They all seem to have been made, as far as any record shows to the contrary, at the same rate of revolutions or alternations per time unit. Transmission of stress in steel or iron requires a certain time, like sound or electricity. How much time is not definitely known; but it may reasonably be assumed that a small dislocation of the molecules requires less time than a large dislocation, all within the limits of elasticity of the material. An oscillation of stress, as in hard steel, from 46 500 lbs. tension to 46 500 lbs. compression, that is, 93 000 lbs. between extremes, as was the case in the tests mentioned, will require more time than an oscillation, as in soft steel, from 26 000 lbs. tension to 26 000 lbs. compression, or 52 000 lbs. between extremes. The modulus of elasticity being nearly the same in hard and soft steels, the stress in the hard steel oscillated, as it were, through an arc 1.8 times longer than that for soft steel. Mr. Lindenthal.

The spindles of hard steel should have revolved slower in proportion, as it required proportionally more time for recovery from stress than did the soft steel at the same rate of recovery. The tests would then have been on the same basis for comparison. For both steels the proportion of ultimate stress to the calculated working stress, called by Sir Benjamin Baker the "factor *A*," was the same. As the rate of revolutions was alike for both steels, the harder steel was comparatively overstrained and likely to give out first, and it did. There is no means of judging whether the hard and the soft steel were not both overstrained. Alternations at slower and faster speeds and with same stresses would be necessary for comparison. Additional tests, systematically made, are needed on this question before a correct judgment can be arrived at. There is, however, nothing in the nature of hard steel *per se* which would make it a less enduring material than soft steel for the slow and gradual alternations of stress in a stiffening truss, which may never have to resist the maxima of the calculations.

The time interval between the alternations would be about 20 seconds for very fast trains at 50 miles per hour. The maxima would require six heavy freight trains abreast at that speed, weighing 8 400 tons. As this mode of loading is out of the question, and considering that the stiffening truss is not as vital to the safety of the bridge as the cables or towers, the writer is of the opinion that 95 000 to 100 000-lb. steel, which is also remarkably tough, as proved in wire drawing, with an elastic limit of 56 000 to 60 000 lbs., is the preferable material, and that 35 000 lbs. tension or compression per square inch is an entirely permissible stress for the chords, from combined live load and temperature maxima, and two-thirds that stress for the web members.

With hard steel the weight of the stiffening trusses in the design could be reduced from 14 340 lbs. to 8 400 lbs. per lineal foot of bridge, the dead load, including cables, from 39 000 lbs. to 32 000 lbs. per lineal foot, and the cost of the bridge would be less by $1 400 000. This is

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Mr. Lindenthal.

for the same deflections, 3.5 ft. at quarter, as are assumed by the author. For the greater deflection of 4.35 ft. the cost would be reduced $460 000 further.

The shop work for the heavy sections, in which all rivet holes must be drilled, will be no more expensive in hard steel than for medium or soft steel.

The reasons given for the use of nickel steel in stiffening trusses do not seem to warrant the large extra expense. The quality of toughness does not come into action in the chords of a stiffening truss, but the quality of elasticity does. Hard steel is the more elastic material. The range of the elastic limit between extremes in hard steel is over 120 000 lbs., against only 72 000 lbs. in medium steel. If the latter were preferred for the greater resulting dead weight and greater inertia of mass in the bridge, then the same advantage, if it be any, can be obtained more cheaply with lighter trusses of hard steel, and with stone ballast at $1 per ton in place of the $80 per ton for steel.

The suitability of hard steel is proved by the arches of the St. Louis Bridge, which for 22 years have been strained nearly 30 000 lbs. in compression, and 20 000 lbs. in tension, per square inch alternately many times a day, without showing the slightest defect. Much better and more uniform steel than that can now be had at a low price. The similar daily strains in the stiffening trusses of hard steel would be 8 000 lbs. tension or compression.

No severer test of hard steel can be had than that of steel rails, which are subject to continuous reversion of strains and to the shock and peening effect of rolling loads.

If nickel steel can be had cheap enough, it would be the material of all others for the cables, for which the quality of toughness is indispensable.

For the thin sections of the floor construction, the usual soft steel is preferable, less for its supposed toughness than for the cheaper shop manipulation of punching and reaming, instead of drilling, the rivet holes.

In the proportioning of the wind bracing, for which the chords of the stiffening trusses act also as the wind chords, the author seems to assume that the lower and upper wind trusses would act together, but this is doubtful. In the absence of adequate cross-bracing one wind truss may deflect sideways more than the other. The maximum side deflection is given as 8.75 ft. One wind truss may deflect 3 ft., the other only 2 ft. A wrenching of the web members is liable to occur where the middle strut is attached. The same wrenching will happen when one stiffening truss deflects vertically more than the other. It would seem better to have no cross-bracing at all than the one proposed. A strong cross-bracing is an advantage, if so designed that the wind trusses

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must deflect together, and so that it will not injuriously affect the web members. Mr. Lindenthal.

The relieving of the towers from transverse strains by spreading the backstays would seem a somewhat costly device where land is expensive, and it is not necessary for stability. On the contrary, a strong cross-bracing between the towers is conducive to greater lateral stability. The towers should stand on straight lines at right angles to the axis of the bridge. The backstays can be spread from them also, if desired.

The fact seems to be that the spreading backstays and twisted towers are for the accommodation of the cable connections on top of towers, which is a point that should count against them. The towers should stand at right angles to the axis of the bridge, because the tower tops must deflect in vertical planes parallel to this axis. They cannot deflect, theoretically, in the planes of the spreading backstays with the brace between their tops. It is the more rational arrangement, although it would require an entirely different kind of top bracing, the extra expense of which would be insignificant. The unsightliness of large towers, twisted with reference to each other for so trivial a reason, would hardly appear excusable to the beholder without an explanation which the odd appearance of the structure itself would not suggest. The base of the Eiffel tower furnishes a sample of construction for meeting temperature strains from deep cross-bracing.

No practical advantage results from having the cradled cables further apart on the top of the towers than at the middle of the span. On the contrary, it should be avoided. Should the wind trusses deflect sideways 8.75 ft., given as an extreme by the author, the tower posts and outer cables on the wind side and the inner cable and tower posts on the lee side would have to sustain a larger proportion of load, amounting to an average for the entire length of 4% of the suspended dead load. A better arrangement would be to have the two cables in parallel cradled planes, or one above the other; or, better still, to have one with one-eighth or one-tenth deflection and the other below with one-twelfth deflection in the same cradled plane. It can easily be arranged for both cables to bear accurately one-half or any other fixed proportion of the load at all times, with the great advantage added that cables of different deflections neutralize the tendency to vibration or tremors from moving loads, as distinguished from deflections, a useful office, which in the Brooklyn Bridge, for instance, is incidentally fulfilled by a network of stays.

The most remarkable feature, and the most important also in the author's judgment, is the system of cables composed of coiled-wire ropes, selected on account of readier erection. The ropes are cut over the towers and fastened to them with sockets. The vital importance of this detail to the safety of the structure invites special attention.

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Mr. Lindenthal.

In all tension members for bridges it is held to be essential that on testing they should pull apart between the fastenings, in eye-bars between the eyes, and in rods with upset screw-ends in the body of the bar. In a large suspension bridge, the case of all cases, the same rule ought to hold true, if possible, for the cables. The writer believes with the author that no socket fastening can be made which will break a wire rope between the fastenings. Certain mechanical reasons would at least so indicate, and they are corroborated by the fact that all attempts in that respect have so far failed. Until they succeed, however, cables of wire ropes, having socket fastenings on the towers, should not be thought of for such an important structure. The supposed readier erection will not offset the great disadvantage of having the weakest part of the cable just where it ought to be strongest.

The method proposed for connecting the cables to and on the towers is open to grave objections.

In the first place it will be noticed that the ropes would spread out, beginning at a point 50 ft. from the tower, where the cable would have a diameter of 36 ins. to a diameter of about 7 ft. on the towers. The ropes, supposed to be adjusted to equal tension for a middle temperature, are ordinarily compacted into the round cable form before the superstructure is suspended from them, and it is so assumed by the author in this case. The ropes must hang in vertical planes. In that position they will not all have the same length, because of the twisted towers and the twisted end connections. Each rope will have a different length, which must be adjusted on the towers, so all the ropes shall be strained alike; otherwise, they would have different deflections and could not be compacted. When compacted, the friction will prevent the ropes from sliding past each other, a condition which contributes to stiffness, and is one of the advantages sought in a compacted cable. It is not material, whether the compacting is by means of wire wrapping or by means of clamps, additional or not, to those for the suspenders.

In a compacted cable, having near the towers an enlarged diameter, as in this design, inequality of strains in the ropes from that cause is unavoidable. The upper ropes having to carry part of the weight of the lower ropes will be strained more than the lower ones. It may be taken for the present as a negligible inequality. But the uneven tension in the ropes near the towers caused by swinging the cables from the vertical into the cradled position, and the uneven tension caused by the vertical deflections of the cables, are not negligible quantities. The uneven tension from cradling cannot be prevented by pin bearings set at right angles to the horizontal projection of the tangent of the cradled cables, when, after all, they are to be erected and compacted in vertical planes. The inequality in the tension of the ropes from cradling will depend upon how much the friction

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in the compacted cables interferes with torsional sliding of the ropes upon each other. At a low estimate, it may require the outer ropes to extend 1.3 ins. in 50 ft. near the towers in the outside cable. The resulting extra strain from the bending of the cables would be 58 000 lbs. per square inch in the outer ropes, or more than enough to relieve the ropes on the opposite side of cable for 50 ft. from any strain at all. This is on the supposition that the cables have been compacted in vertical planes and for their entire length, except 50 ft. from each tower. Mr. Lindenthal.

Turning for a moment to the vertical deflection, the weight of the superstructure will be found to cause the cables to deflect 10 ft. Taking the case of a hot summer day, when the cables would drop another 3 ft. at the center, combined with heavy trains on the half span, producing at the quarter a further deflection of 1 ft., it will be found that by reason of the change of angle formed by the tangent to the cable curve and the tower, the upper ropes would stretch in 50 ft. as much as ⅛ in. more than the ropes in the middle of the cable. Again, on the assumption that the cables are compacted for their entire length except 50 ft. from each tower, the excess strain on them would reach 33 000 lbs. per square inch, or more for greater deflections in the quarter. The ropes on the opposite side of the cable would be strained 33 000 lbs. per square inch less. A diminution, or possibly a reversal, of these excess strains, but much smaller in amount, would take place in the upper and lower ropes on very cold days, at the end of the unloaded half of the cable.

While these excess strains, under the conditions named, when added to the strains from dead and live load, would possibly yet be within the elastic limit of wire ropes, they indicate how liable to overstraining the cables as proposed would be at the point of their greatest weakness, namely, in the socket fastenings at the towers.

A diffusion and great diminution of the unequal tension in the ropes could be effected by pulling the cables into their cradled position at the same time that they are compacted, commencing at the middle of the span, and also at the same time suspending the floor, commencing at the middle and proceeding symmetrically towards both towers. As a further precaution, the cables should not be compacted between the tower and the first suspender, 200 ft. away. This method of erection is not contemplated in the design of the author; but if not done, it may happen that over 10% of the ropes at the towers would be strained to nearly two-thirds their ultimate strength, and only a small number in the core of the cable would be free from excess strains.

It might be urged that the same objection would apply to cables continuous over the towers. So it would, if not guarded against. It is entirely practicable with continuous cables to reduce the excess

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Mr. Lindenthal.

strains from bending at the towers to a small and negligible amount, even with very large cables. The two cables of 36 ins. diameter could very well be combined into one of 50 ins. diameter. It is not possible, however, with cables cut over the towers.

The cradling of the cables is too valuable a feature to omit on account of bending strains at the towers, which can be fully met for the largest cables, notwithstanding that they be compacted when hanging in vertical planes. Special precautions in the erection of wire cables have not appeared to be necessary before, because the largest so far made were those for the Brooklyn Bridge, 15½ ins. in diameter, and the bending strains in them at the towers do not exceed 3% of their ultimate strength, as against the possible 18% to 30% under the same conditions for the cables fastened to the towers and erected on the plan proposed by the author.

The second important objection to the author's plan is that the cable connection on the towers is itself not without serious uncertainties. During the erection of the cables only the lower tie plates can be in place. They are arranged to take up in each set the entire pull of one cable from its own weight. This pull is 8 400 000 lbs. per cable. The upper and lower ties are supposed to act together in resisting the maximum pull of 40 000 000 lbs. in the inverse proportion of the distances of their respective center lines of tension from that of the cable, which is about as 2 ft. 8 ins. to 7 ft. 6 ins. on the scale of the detail drawing.

The pull in the lower ties from the cable alone, 4 100 lbs. per square inch of gross section of 2 052 sq. ins., will extend them (in 17 ft. 6 ins., for $E = 28\,000\,000$) 0.03 in., which is about twice the least necessary amount of play of a 5-in. pin in the pinhole for purposes of erection. After the upper ties are inserted, and before the play in the pinholes at both their ends can be taken up, the lower ties must further extend twice 0.03 in., which means that they will take up 16 800 000 lbs. more, or a total of 25 200 000 lbs., of tension before the upper ties can come into action. The result would be that of the ultimate 40 000 000 lbs. tension, the lower ties would take up 35 870 000 lbs. and the upper only 4 130 000 lbs., instead of the 29 508 200 lbs. and 10 494 800 lbs. respectively given on detail drawing.

The strength of the nine-bolt connection, set in diamond form in the lower ties, cannot be accurately estimated. All that can be said is that it is probably better than the square form; but even this may be disputed by some engineers, from recent tests made for purposes of comparison with riveted connections where the additional holding force of friction occurs. In the bolt connection it would be absent. To be on the safe side, it will be best to deduct three pinholes from the gross section of 2 052 sq. ins., thus giving a net section for the lower ties of 1 410.75 sq. ins. For a play in the pinhole of 0.027 in.,

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which for first-class workmanship is yet a usual allowance for 5-in. pins, the maximum pull of 40 000 000 lbs. would be taken up entirely by the lower ties. The upper ties would then be superfluous. Mr. Lindenthal.

With the most favorable conditions and highest class of machine work, the stress per square inch from 35 870 000 lbs. could therefore be confined to 17 500 lbs. for the gross section, and possibly to 25 400 lbs. for the net section. The latter cannot be regarded as a permissible stress for such a vital connection, which is further subject to large but indefinite bending strains from the holding-down force of the four vertical ties, and from the unequal tension in the cable ropes before mentioned.

All these strains are very large, and slight dislocations, as may be seen, will affect their distribution, so as to produce serious overstraining, amounting to unsafety of some parts.

A remedy might suggest itself in the warming of the upper ties, perhaps with steam pipes, for lengthening them when inserting the three pins at one end, which all must go in at the same time, if they are to resist equal amounts of tension. But whoever has tried the operation with only six eye-bars and one pin will not advisedly propose it for nineteen bars and three pins as an erection device, which, if perchance successful, would introduce obviously only new uncertainties. The only safe way would be the erection of the upper and lower ties at the same time, but that would make the erection of the socketed ropes impossible.

What are, then, the supposed advantages of cutting the ropes over the towers and fastening them there? The author mentions three.

The ropes can be made at the shop, adjusted there to exact length, readily transported to bridge site, and then erected in the least possible time.

Next, avoidance of the decided turns required over the tower saddles, and the readier use, therefore, of a strong stiff wire.

Last, the possibility of using steeper, and, therefore, heavier, backstays to anchorages nearer the towers if necessary, independent of the cable section in the main span.

As regards the first point, the same advantage, in a greater degree, can be had with ropes continuous over the towers, and reaching in one length from anchorage to anchorage, thus avoiding on the towers at least four weak places. The length and weight of such ropes would not be too large for transportation. The adjustment at the shop would be worthless unless the rope is put under sustained tension equal at least to that it would have in the bridge from dead load alone. It can be most easily done in both cases with the ropes hanging on the towers.

The method of erecting the cables on iron horses, supported on temporary ropes with men standing by, and putting each rope into

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Mr. Lindenthal.

its exact position, can be used equally well for cables continuous over the towers.

Making the wires of one length from tower to tower would be no particular advantage. The splices of, say, fifty wires composing a coiled rope can readily be so distributed, that in any part its strength would be diminished not more than 2%, which is less than the variations occurring in the strength of the wire itself. The friction with which a wire is held in a rope, or even in a wrapped straight-wire cable, is so great in the length of a few feet that it could not be pulled out without breaking.

On the second point, it may be observed that for continuous ropes the flat curve over the tower saddles can be of no possible harm. The wire ropes must be in any event transported, wound around drums of smaller diameter than the saddles on the towers would have. The author himself proposes to use the same ropes for suspenders bent over cable saddles only 36 ins. in diameter.

There is no necessity for preserving the round section of very large cables over the towers. The better plan is to gradually change from the round to a rectangular flat section, which would afford a better bearing on the saddles. The elaboration of such a detail, designed by the writer for another occasion, is here not necessary.

The third point, of separate backstays for steeper inclinations and heavier sections than in the main span, the writer had occasion to consider in connection with an important bridge in Europe on which he was consulted. He is prepared to say, that the increase of section in the backstays, consisting of wire ropes or straight wire, is entirely practicable, without special fastenings on the towers, and without disturbing the continuity of wire ropes or straight wires over the towers.

The precautions in the erection of cables over flexible iron towers (and all towers are flexible) will be necessary not less with cables cut over the towers than with continuous cables, which can likewise have a fixed bearing on them. Care will have to be taken in both cases that under the full load of the superstructure the towers shall have no bending strains in them for that neutral position.

Cables of wire ropes do not offer sufficient advantages to make them preferable to cables composed of strands of parallel wires, the fastening of which to pins by means of looped ends around shoes has been found by experience to be a safe and convenient detail, and a better one cannot be devised. The slow work in spinning the cables for the Brooklyn Bridge is no criterion for similar work elsewhere; it can be done much faster, and it is believed at a price 25% below the price of the coiled wire ropes.

The suspender saddles, as shown on the design, would not be secure against slipping on the steep part of the cable, if the writer may trust

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his experience with similar and proportionally stronger clamp details on wire cables only 6½ ins. in diameter. On larger cables, the difficulty of getting a tight grip on the cables is, of course, much greater. Mr. Lindenthal.

It will sometimes be an advantage to continue the stiffening trusses to the anchorages and suspend them from the backstays. The saving of the foundations for intermediate piers between towers and anchorages may more than pay for the extra cost of suspended trusses. The bridge receives not only a better appearance, but there is a gain in rigidity for the main span, provided such an arrangement is not forestalled by local conditions.

It is gratifying to the writer, that the author has arrived at the same general view he held and promulgated long ago, namely, "that a great suspension bridge, which would be well adapted to railroad service, would involve no insurmountable difficulties of construction," and at an estimated cost, the total of which does not greatly differ from that of the writer. That he does not agree, however, with all the views of the author, this discussion sufficiently indicates.

The author was also preceded in his studies by two commissions of engineers investigating the same subject. One of the commissions, consisting of five eminent expert engineers, including the author, arrived at conclusions and estimates more than ordinarily conservative, naturally so in view of the novelty and importance of the problem. The amount of study and labor within the short time allowed to the commission must be designated by those understanding the scope and difficulty of the work as likewise more than ordinary and thorough.

The work of the second commission, consisting of three officers of the Corps of Engineers of the United States Army, one of whom was chairman of both commissions, covered a broader field, and came nearer the truth as regards quantities and cost. Their report contains an original and comprehensive discussion of the leading principles in the construction of suspension bridges and of the derivation of construction weights, and the most complete compilation of data on wind pressure published in any language.

The accumulating literature on the subject of the suitability of the suspension bridge for railroad purposes, and, in fact as the only practicable type for the very longest span, is furnishing valuable data for further study. Indeed, an engineer investigating the problem of a North River bridge for actual construction would not do his full duty if he rested satisfied with the investigation of only one system of suspension bridge, and that the simplest and most obvious in form. The writer had long ago gone further, and spent much labor in examining other systems. It was not only interesting, but necessary, to make comparisons of the different systems, and with the information so collected, to decide upon that system which for the given conditions would answer best.

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Mr. Lindenthal. It may be remarked that Fidler's system of double cables, intersecting at the middle of the span and near the towers, with the particular arrangement shown in plan No. 3 of the published preliminary designs for the Forth Bridge, showed temptingly great advantages. It is very economical, and can also be made architecturally pleasing, but the erection of this system with wire cables presented difficulties which do not occur with the braced double-cable system selected by the writer for his design. It took some study and time before the details were entirely satisfactory; but they are so now. A detailed description of the design would of itself be a long paper, which may not be presented here as a part of this discussion.

Mr. Hildenbrand. W. HILDENBRAND, Esq.—The following remarks are confined to that part of the design which the author considers to be radically different from suspension bridges hitherto built. He would have been more precise in saying, different from large suspension bridges hitherto built, because the same general principles that guided him in his design have been and still are applied to small bridges. When John A. Roebling and his son, Washington A. Roebling, the builders of the first large suspension bridges, abandoned those features and substituted others, it was with the object, according to their opinion, of improving in expediency and economy the methods of cable-making.

There are three distinct features in which the author's cables differ from those of the present large bridges:

First.—The cables are composed of twisted wire ropes in place of straight wire strands.

Second.—The cables are immovably fixed on the towers in place of resting in roller saddles.

Third.—The cables are not continuous, but consist of separate lengths for each span.

The first two features are common to most small bridges; the third feature has not frequently been applied.

Wire-Rope Cables.—The author gives the reasons why cables made of wire ropes have been discarded for large bridges, viz., because wire ropes have proportionally less strength than straight wire, and because no attachments can be made with any reliable uniformity in strength. The difference between the strength of a wire rope and the same section of straight wire has never been expressed in a formula. It is only known that it varies from nothing, which is an exceptional occurrence, to 15%, which is a frequent occurrence. In the wire-rope list of John A. Roebling's Sons Company the strength of the ropes is rated about 12½% lower than what it should be theoretically, and 10% has generally been accepted as a fair average.

The fastening of ropes in sockets is a still greater factor of uncertainty. Ropes can and have been socketed developing the full strength of the rope, but it requires large, clumsy sockets and extreme care and

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skill in putting on the socket, such as may be exercised in experimental cases. It would not be practical to rely on perfect workmanship, but allowance must be made for the greater or lesser skill of the operator. Mr. Hildenbrand.

Under ordinary circumstances and care the socket strength is frequently not over 75% of that of the rope, while a strength of 85% may be considered a good average. This deficiency of 15% added to the above-mentioned loss of 10% makes a total loss of 23½% in the strength of a wire-rope cable as compared with a straight-wire cable. For the sake of argument, the writer will make an extra allowance of 8½% for special care in the manufacture of rope cables and for losses in wire cables on account of kinks and splices in the wire, and call the loss of relative strength between the two kinds of cables but 15%, which is a liberal assumption in favor of rope cables. This will give the following comparative cost for the case of the author's bridge:

58 553 000 lbs. of wire ropes at the minimum price of 5½ cents per pound.....	$3 220 431
58 553 000 lbs. — 15% = 49 770 000 lbs. of wire at a maximum price of 3¾ cents per pound,	1 866 375
Difference	$1 354 056

The writer believes that a saving of $1 350 000 alone in the cable material, not counting the costly work of socketing, would in most cases decide against the adoption of wire ropes, unless there be other and greater advantages connected with the latter to counterbalance this loss.

Fixing the Cables to Metal Towers.—This can be done safely, as the author has demonstrated by calculating the ordinary motions under passing loads and under changes of temperature, and comparing them with the flexibility of the towers. It seems to the writer, however, that this feature has been chosen unfortunately in connection with another feature of the design, viz., the omission of suspenders in the land spans. When there are suspended land spans, the cables are balanced at each stage of the erection, but when the land cable must first balance the empty, and afterwards the loaded, center cable, it is necessary to give the land cable, when first put up, a greater length than it will have in the finished bridge. For the author's bridge, the writer calculated this difference of length to be 3½ ft., or, better expressed, it requires a motion of 3½ ft. on top of the towers to balance the spans before and after being loaded with the dead weight. Is the author prepared to allow this motion to the towers? It would be much simpler to rest the cables in roller saddles, place the latter 3½ ft. out of centers, and let them gradually roll into their final position.

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Mr. Hildenbrand.

Of course, the difficulty could be obviated by temporarily supporting the land cables, which is frequently done in small bridges; but this would prove to be an expensive "extra" in a bridge of such large dimensions and where the land cables, fully or partially, cross a deep river.

3. *Discontinuity of the Cable Over Three Spans.*—This is a feature which, naturally, has not often been applied or suggested. Engineers, generally, try to avoid splices and use the longest available lengths of bridge members; hence, when it is just as easy to get such a member, a wire rope or wire strand, in one length, it seems injudicious to cut it deliberately into three pieces and splice them again, unless this be justified by strong reasons and decided advantages.

In the case of the new Danube Bridge at Budapest, where the same feature of socketing the cables on top of the towers was proposed, the designers considered it justified by the difference of tension in the land and river cables which, if made continuous, had necessitated a considerable waste of cable material. No such reason exists in the author's design, but he states for his reason the two following advantages: that it would shorten the time of erection, by obviating the manufacture and regulation of wire strands, and that it would avoid bending the wires over saddles and end pins.

This second advantage can at once be dismissed, because it is absolutely of no disadvantage for the wire to be bent over saddles and around the end shoes.

The first-claimed advantage looks very plausible, but it is based on an improbability and a misconception of the different phases of wire cable-making.

The improbability consists in the author's belief that 253 wire ropes, each 3 300 ft. long, could be cut to precise lengths and manufactured with such homogeneousness, that all ropes of the same length would retain an equal length if suspended with the same deflection. That this is not absolutely possible needs no proof, because there are no two things alike in the universe, but that it is not even practically probable a few figures will demonstrate.

Each rope is supposed to weigh 10 lbs. per foot, contains 3 sq. ins. of metal, and has a length of about 3 333 ft. if suspended with 400 ft. deflection. The stress in each rope, due to the dead load, will be 70 tons. When the rope is first suspended, its tension will be about 18 tons, or 12 000 lbs. per square inch. The author records the result of some tests, showing that the elongation in 200 ins. of one sample of rope for a tension of 12 000 lbs. per square inch would be 0.1671 in., and in another sample of rope 0.2012 in., being a difference of 0.0341 in. in 200 ins., or of 6.82 ins. = 0.57 ft., in a length of 3 328 ft. This difference in length would cause a difference in deflection of 0.862 ft. = $10\frac{3}{8}$ ins. Such a discrepancy in deflection would not be considered a good

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regulation, and this would be the very best that could be accomplished according to the apparently favorable tests quoted by the author. In reality the case would be much worse, because the difference of elongation between two ropes, if measured in a length of 3 000 ft., would be much larger proportionally than if measured in 17 ft., on which the above calculation was based. Considering also the changeable specific gravity of the metal, the difference of temperature between different ropes when cut to length, the impossibility of observing accurately the temperature or of measuring the length, and the difficulty of putting on sockets to occupy a precise length, it will be seen there are many more factors for causing the ropes to hang at different heights, if the variable modulus of elasticity, as proved above, should not be sufficiently convincing. In other words, it would never do to suspend the ropes indiscriminately with blind faith in shop accuracy, but it will be necessary to regulate them relatively. This being admitted, all supposed advantages of socketing are gone, and nothing is left but the disadvantages of inconvenience, loss of strength, loss of time, and useless expense. Mr. Hildenbrand.

The advantage claimed for the rope cable, that ropes can be brought ready made to the bridge site and save the time for spinning a wire strand, is based on the belief that the latter work requires a very long time. This is erroneous; generally, there is more time consumed in regulating a strand than in making it. A wire strand as strong as one of the ropes in the author's cable would consist of ninety No. 6 wires, which could easily be laid up in one day, giving 12½ minutes for a trip of the wire wheel and 12½ minutes for regulating two wires, which is ample. The author believes he could haul three ropes over and put them in place in one day, but according to the experience gained at the Brooklyn Bridge, where a number of ropes were hauled over for temporary purposes, he will do well if he puts one rope in place in two days. He may possibly do a little better than this, but in no case does it appear that he would gain much, if any, time over that needed for making a wire strand of equal strength. On the other hand, the regulating apparatus on top of narrow towers will be much less convenient than if placed on the spacious anchorages. This, and similar other points in cable-making, will become more apparent when "the various special appliances are worked out" to which the author finds it occasionally necessary to refer in his paper.

Taken as a whole, the author's plan could creditably be executed precisely as it stands. There is nothing in the design that could not be sustained theoretically, or be built practically, and the details have been admirably worked through with minuteness and on scientific principles; but from what the writer has tried to show by figures and arguments, he would venture to predict that the cables for the next large suspension bridge will not be constructed on the author's present plan.

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Mr. Hildenbrand. There are other features in the proposed design with which the writer fully agrees. Among these are the omission of suspenders in the land spans, the omission of suspenders for 150 ft. out from the towers, the construction of the stiffening truss as a cantilever for 150 ft. out from the towers, the anchoring of the stiffening truss at the towers, and the continuous extension of the stiffening truss over the land spans. All of these features were adopted by the writer in the reconstruction of the Covington and Cincinnati suspension bridge, which is at present in course of construction. In a few months from now it will practically be demonstrated whether these latter features are judicious applications or not.

Maj. Raymond C. W. RAYMOND, M. Am. Soc. C. E.—This paper is of special interest in view of recent investigations with reference to the construction of suspension bridges of very large span. In August, 1894, a board of bridge engineers appointed by the President of the United States* submitted a report upon the length of span, not less than 2 000 ft., which would be safe and practicable for a railroad bridge to be constructed over the Hudson River at New York, in which report the subject of suspension bridges of large span received special consideration. A month later, a board of officers of the Corps of Engineers, appointed under the instructions of the Secretary of War, submitted a report upon the maximum span practicable for suspension bridges. The calculations and investigations of both boards were necessarily approximate in character, owing to the limited time available for the preparation of the reports. The author was a very active and valuable member of the board first mentioned, and the writer, who was a member of both boards, considers it of great importance that this detailed study practically confirms the conclusions of both boards in all essential matters.

As the author points out, the suspension bridge has been generally superseded by truss and cantilever bridges for all except very large spans. The cases requiring suspension bridges will hereafter be rare, and the engineers who design them will not have a practice founded upon extended experience upon which to base their calculations. This fact gives an increased importance to conclusions arrived at by capable and experienced bridge engineers in the few cases studied in detail.

As was found by the Army Board, it is almost impossible to discuss the theory of the suspension bridge satisfactorily in a general way, because the problem is essentially one of details, and the details depend largely upon accidents of location and the relative cost of shop-work and the materials of construction. The method adopted by the author of taking a concrete case and working it out in detail is undoubtedly the only one which can give definite results. The case selected is the one approximately investigated by both the New York

* Under the Act of June 7th, 1894.

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Bridge Board and the Army Board; and although the location chosen is higher up the river, it does not differ essentially in length of span from that discussed by Gustav Lindenthal, M. Am. Soc. C. E., in his elaborate and detailed design for the North River Bridge at Twenty-third Street, which design was completed several years before the commencement of any of the investigations previously referred to. Two studies of this problem were made by Mr. W. Hildenbrand for the New York Chamber of Commerce, and are published as appendices to the report of the New York Board. Although these studies were of a preliminary character, they deserve careful consideration on account of their author's connection with the erection of the East River Bridge and his long experience in wire construction. Recently L. L. Buck, M. Am. Soc. C. E., prepared detailed plans for the new bridge across the East River, which will be of great interest, although the span is only 1 600 ft. Maj. Raymond

While it is not possible to make a full comparative study of these different investigations within the limits of this discussion, it may be of interest to point out some of their principal agreements and differences.

Perhaps the most important point in connection with these investigations is the claim which has been made that the suspension bridge differs so much in structural character from other bridges that the rules usually adopted for unit stresses and safety factors require modification in this case. The writer believes that this has been fully stated by Mr. Lindenthal and recognized in his computation, and this view was also adopted by Mr. Hildenbrand in his designs. It was also recognized in the report of the New York Board.

The Army Board devoted considerable attention to this question. It remarked that "the great distinction between the stable equilibrium of a suspension bridge, which cannot break down from the failure of any stiffening member, and the unstable equilibrium of a truss, arch or cantilever bridge, in which the failure of a member may involve the collapse of the entire bridge, ought to receive full recognition in the adoption of unit stresses and safety factors." Again, the Board remarked that "rigidity is in this case of much less importance than it is in most other kinds of bridges; indeed, it may be shown that a certain small flexibility is a positive advantage in suspension bridges;" and still again, "the Board does not doubt that within narrow limits a certain degree of flexibility is an advantage to the bridge. Deflections in a system of stable equilibrium do not impair the safety of the structure as they do in an unstable system like the upright arch, and they may exert a very beneficial influence in modifying the dynamic effects of a rapidly varying live load."

It gives the writer great pleasure to find these views, which were by no means formerly accepted by all competent bridge engineers, fully stated and confirmed in the paper under discussion.

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Maj. Raymond. In accordance with these views, the Army Board adopted for the cables a working stress of one-third the static breaking load, and this is a greater working stress than has usually been adopted in the cables of suspension bridges of ordinary span. The same stress was adopted by Mr. Hildenbrand and also by the New York Board for comparative purposes. The Army Board gave in full its reasons for adopting this increased stress. The author has practically adopted the same working stress.

For the stiffening girder the Army Board adopted 15 000 lbs. per square inch for the working stress, while the New York Board limited the stresses due to a moving load to 12 500 lbs., but allowed the stresses from the combined effects of moving load and wind to run up to 22 500 lbs. The Army Board remarks that "the only duty of these girders is to distribute the live load and thus prevent inconvenient deflections. It is not necessary to give them the margin of strength which they would require if they were essential to the stability of the bridge."

Mr. Hildenbrand adopted a working stress of 20 000 lbs. per square inch in his first design, but in his second design reduced the stress to 15 000 lbs. at the request of the New York Board. He believes that 20 000 lbs. will give ample safety because the reverse strains will occur only at long intervals.

The author proposes to use nickel steel in the construction of his stiffening girders and adopts a working stress of 17 000 lbs. per square inch. He remarks that "a suspension bridge must be permitted to change its shape within proper elastic limits, and this change of shape must be made the basis of calculations in proportioning the structure." In the writer's opinion, this simple statement embodies the only rational theory which can be followed in the designing of suspension bridges.

The writer does not know exactly what working stresses were adopted by Mr. Lindenthal for his girders, but an examination of his design leads to the belief that he would not consider the stresses adopted by the Army Board as too great.

As has been before remarked, the suspension bridge problem is essentially one of detail. Perhaps the most important and troublesome detail in the whole problem is the method to be adopted for transferring the stresses and providing for changes of form over the tops of the towers. It is believed that no method has been employed or proposed which is not open to some objections. The author proposes an entirely new system, fastening the ropes forming his cables in sockets at the tops of the towers and taking up changes in the length of the backstays by the motion of the towers alone. This modification, as the author remarks, is really the essential feature of the whole design, and he states the objections to the system and its advantages. There

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are two important particulars in which the adoption of this system appears to have modified the general design of the structure—the dip of the cable curve and the character of the vertical bracing. Maj. Raymond.

The author adopts one-eighth of the span for the dip. This was the value adopted by both the New York and Army Boards. The writer believes that a smaller dip would be decidedly more economical, if the fastenings are left out of consideration. A decrease in the dip requires more metal in the cables, but less metal in the stiffening girder. In Mr. Lindenthal's bridge the dip is one-tenth, and Mr. Buck appears to have taken about one-ninth. Mr. Hildenbrand adopted one-tenth in his first, and one-eighth in his second, design. In the author's design an expensive material is used in the girder, and therefore it would seem economical to decrease the weight of the girder and increase the weight of the cables by diminishing the dip. Doubtless the author has adopted the larger dip in order to make his towers sufficiently flexible for the purposes of his cable fastenings.

For vertical bracing, the author adopts the straight continuous stiffening girder. Both of the Boards and Mr. Hildenbrand adopted a straight girder hinged in the middle in order to simplify the calculations. The writer believes that the girder hinged in the middle has no advantage whatever over the continuous girder. Even its supposed advantage in the easy determination of the stresses exists only when the computations are of an approximate character, and vanishes entirely when the temperature stresses in the half arches and the details of the middle hinge are considered. If a straight stiffening girder is employed it is considered beyond question that it should be continuous; if for no other reason, because the portion of the live load taken up by the cable is greater with the continuous than with the middle-hinged girder. Mr. Hildenbrand remarks that the weight of the girder will be reduced nearly 10% by making it continuous. Such a girder is adopted by Mr. Buck in his recent design. Looking at the question, however, from a purely theoretical point of view, and leaving out of consideration the arrangements of the cables at the towers and other details, the writer believes that the best place for the principal vertical bracing is in the suspended arch itself. It is true that the longitudinal girder is needed as a floor stiffener, for the support of the lateral bracing and for side guards; but it is nevertheless believed that the same amount of rigidity could be obtained with less weight of metal if the principal bracing were placed between the suspended cables. Mr. Lindenthal has done this in his design. He combines the two systems (which is perfectly practicable since they deflect with the same rate and law), but apparently requires very little from his longitudinal girder, depending almost entirely upon the bracing of his arch to control the changes in the form of his cable curve. With the method of

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Maj. Raymond. cable fastening adopted by the author, the employment of the arch bracing would seem to be impracticable.

There are many other points in this admirable paper which the writer would like to notice if time and space permitted, but these must be left to others. In conclusion it will be of interest to compare the various estimates of cost which have been prepared for a suspension bridge at the locality considered. The estimate of the New York Board for its "Lighter Structure," when corrected to correspond to the "upper location," is about $26 000 000. This estimate, however, was made for comparative purposes, and the Board was properly conservative as regards prices and working stresses. If the cost of structural metalwork were reduced from 4½ cents to 4 cents per pound, and the cost of wirework from 8 to 7 cents per pound, the estimate would become about $24 340 000. Mr. Lindenthal estimates the cost of his bridge at about $21 000 000, but reduces his estimate to about $17 800 000 for this locality. Mr. Hildenbrand's estimate for his second design is about $15 901 000. The Army Board named the sum of $23 000 000 as a reasonable estimate for the cost of constructing a suspension bridge of this length and character. For the design submitted in this paper, which has been investigated with much more care and in much greater detail than any of the others except that of Mr. Lindenthal, the author estimates the cost at $22 500 000.

Mr. Bouscaren. G. Bouscaren, M. Am. Soc. C. E.—The leading features in the author's study of a long-span suspension bridge for railroad purposes are the composition of the cables of ready-made twisted wire ropes instead of straight wires, their discontinuity at the towers and division into main cables and backstays, their fixed attachment to the towers, the particular form of anchorage proposed, and the cantilever shape of the stiffening truss.

The substitution of twisted wire ropes for straight wires raises the question of uniformity of working stress in the different elements of the same cable, which is of first importance. This uniformity is secured to a certainty in the straight wire cables by the method of construction introduced by Roebling, which consists in adjusting every wire to the same deflection. It does not seem that such adjustment can safely be omitted in this case. After all possible care is taken in the delicate operation of measuring every individual rope and the exact distances between centers of towers, and between towers and anchorage connections, possibilities of discrepancies large enough to require a considerable depth of shouldering between the washer plates and sockets will still exist.

The degree of uniformity in the manufacture of the ropes would also affect the uniformity in the working stress of the cables. The favorable results of the tests made at the Watertown arsenal should be confirmed by a large additional number of similar tests before reliable conclu-

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sions can be drawn therefrom. In this connection it would be of special interest to note the exact lengths of specimens between shoulders to determine whether an appreciable amount of slip occurs in the sockets within the limit of the working stress. Mr. Bouscaren.

The proposed covering of the cables with jackets made of non-conducting material is a very essential feature in cables of as large diameter as proposed. This jacketing would preserve a sufficient degree of uniformity in the temperature of the inside and outside ropes; it would also add very materially to the protection of the cables against rusting by shedding the rain water, and would prevent internal condensation in a large measure; but it should not be relied upon exclusively for protection, especially if it were to be discontinued over the clamps as proposed. The continuous wire wrapping, which has been so successfully used on the cables of the Brooklyn and Cincinnati bridges, should not be omitted. Before wrapping, all the interstices between the ropes should be filled with a waterproof material neutral in affinity and stable in composition. Outside of the question of over-loading, corrosion is the one element which renders uncertain the life of metallic structures. No precaution should be neglected to prevent it in a bridge of such importance, which should possess in the highest degree a character of durability and permanency.

The idea of a fixed connection of the cables at the towers is attractive. It utilizes the stiffness of the towers to lessen the deflection of the span. The principal objection to a fixed connection is the difficulty of obtaining the necessary amplitude of longitudinal motion at the top of the towers during erection without creating excessive bending stresses in the towers. This difficulty can be overcome as suggested by the author, or, preferably, the writer thinks, by meeting the erection stresses with additional metal in the towers. As these stresses are only temporary, an increase of 25% over the normal working units is admissible.

Two objections may be made to the simple and ingenious form of anchorage proposed by the author—first, the temporary lower anchorage might be an expensive affair in the absence of solid rock at a convenient depth; second, the completion of the anchorage masonry for the bearing of the upper casting must be delayed until the completion of the cables, which involves the transmission of considerable pressure through green masonry.

The author's treatment of the question of stiffening is logical and proper. It is quite clear that the long undulations in the cables of such a long span, due to the moving load, are harmless, and that no provision need be made for the distribution of a load smaller than that which would produce an inclination of grade equal to the maximum gradient fixed upon for the convenience of operation. The function of the stiffening truss is, therefore, reduced to the distribu-

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Mr. Bouscaren. tion of the difference which may exist between the actual maximum moving load and the load which would produce the maximum gradient. This consideration allows at once a very large economy to be made in the weight of material in the truss.

The utilization of cantilever projections of the shore span to support the stiffening truss has the advantage of securing continuity without the disturbing effect of possible settlements at the towers.

The proposed use of nickel steel in the chords is an innovation in the right direction, like the substitution of steel for iron fifteen years ago. Nickel steel has been used chiefly for armor plates and for special parts of machinery; it is as yet untried on a large scale for structural purposes. The practicability of procuring with desirable promptness 16 000 000 lbs. of this material, possessing a sufficient degree of uniformity in its physical qualities to bear safely a working stress of 40 000 lbs. per square inch, may be questioned, but there are no reasons to suppose that a satisfactory material could not be procured if sufficient time were allowed for experiments. If, as stated by the author, it can be manufactured now into shapes at an advance of only three-quarters of a cent per pound over ordinary structural steel, it could be introduced with economy in the construction of very large truss spans.

The units of working stress used by the author in the different parts of the structure are approximately the same as those adopted by the Board of Engineers in their report upon the New York and New Jersey bridge, but the carrying capacity of his bridge is less than that estimated by the Board, and the prices used in his estimate are lower than those of the Board, which explain the large difference between the total estimates of cost, as given in the two papers.

Mr. Crowell. FOSTER CROWELL, M. Am. Soc. C E.—In presenting the subject of this admirable paper in the form of a preliminary study of a practical character, the author, it would appear, has invited the frankest criticism and a wide range of suggestions, not only upon the radical and bold departures, or advances, from present recognized features of construction which he advocates, but, in addition, in regard to any other considerations wherein improvement is possible and desirable.

The four chief points of novelty in the design, which are the use of twisted wire ropes in the cables, the fixed attachment of cables to towers, the form of the anchorage connection, and the combination of continuous stiffening truss and its cantilever anchor, together with the various details in connection therewith, have been well discussed. The writer will pass to the importance of providing for the grace and majesty of such a grand bridge, by incorporating in its very structure the forms and lines that shall produce on eye and mind the impressions of a beauty equally grand.

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Mr. Crowell.

It is commonly but erroneously considered by engineers that utility in a bridge design is all that is necessary to render it pleasing to the eye, or, at best, that the Aristotelian principles of beauty, which were "Order, Symmetry and the Definitive," are all that need to be observed. Even the supreme ugliness of the ungainly Forth Bridge is said to be transformed into beauty when one comprehends the functions of its several parts, but few besides engineers can appreciate those functions, and so, to most observers, it is and always will be one of the ugliest as well as most stupendous structures.

It would seem that the author shares the view that in an engineering structure "handsome is as handsome does," for he is silent in regard to the matter of proportion and form, excepting as called for by the strain sheet or the requirements of the laws of stability; in fact, he even introduces intentional irregularities in the plan and placing of his towers, and treats them as matters of no consequence, although he refers to the fact that the proposed twisting of the towers with reference to each other would be manifest, and concludes that the fact of the towers themselves being built out of square would seldom be observed. The only references the paper contains to the appearance of the structure are as follows: "Fig. 15, showing the general elevation and tower, shows the tower as finished, with a room about 50 ft. square on top, surmounted by ornamental work terminating in a flagstaff," and "the ornamental work on the top of the towers, with provisions for lighting, etc., would cost $100 000." It is the writer's present purpose to call attention to the avoidance of purely æsthetic principles in the author's design, not as a distinctive demerit, but because it is a characteristic lack in most designs of modern metallic bridges, large or small, and to urge that it is of the very first importance in developing a design like this, that those principles should be fully recognized.

By this the writer does not wish to be understood as recommending either mere decoration or ornamentation superadded for effect, but that the forms and lines of the parts of the structure wherein a choice is possible should be both expressive of their function and harmonious with the dominating lines, which, in the case of suspension bridges, are the sweeps of the cables.

Nature abhors a straight line as much as she does a vacuum, and it is by the use of all sorts of orders of curves that the impressions of beauty are produced. When, then, straight-line towers are introduced into the design of a suspension bridge, the principles of beauty are rudely violated, and the fact that the lines of floor and stiffening truss are also curved emphasizes the incongruity to the point of ugliness. Furthermore, the towers are not simply shafts, rising to a great and conspicuous height; they are to support a visible weight; their purpose is plainly revealed. They should, therefore, be so designed in out-

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Mr. Crowell. line as to give the right impression of strength continued to the point of support. The cables should not appear to pass through them, as do the cables in the otherwise beautiful bridge in Budapest, but to rest upon them. With the carrying of the cables their function ends, so they should not be carried higher than absolutely necessary, and they should not have finials and other features of ornamentation; especially should they not have flagstaffs. If there are to be observatories on the towers they should be plainly in evidence; all the finish at the top of the towers should be of great simplicity and massiveness. The proportions and lines of the towers should give an expression of repose and superabundant strength combined with lightness. Then, too, they should be of such outline in plan as to eliminate the necessity for being placed unsymmetrically, even at some sacrifice of economy of material and cost.

The position of the rocking bents should be back of the front posts, masked, as far as possible, by the towers, provided this can be done conveniently, which would seem to be the case, for, pleasing as the device, as designed, might prove, viewed as a means to an end, the harmony of the proportions of the towers would be sadly marred.

There is nothing novel in the principle here suggested, apart from its application to bridge design. The value of the curve in architecture and its use for centuries to produce impressions upon the mind of the observer, whether in the outline of a column, or in the proportion of spire or cathedral arch or dome, or in rectifying the crooked effect produced by a straight cornice, need not be cited. One has only to contrast mentally the effect of the Eiffel Tower with the straight line skeleton observatories and electric light towers commonly seen, which have even less beauty than an oil-well derrick. There is, moreover, a utilitarian consideration for employing a curved outline for the suspension bridge tower designed to be flexible under the action of the cables, but that is a refinement that need not here be discussed.

Passing briefly to another feature of the plan, the dropping of the two spans at each end of the structure, *i. e.*, making them deck spans, is an artistic mistake, however sound the reasons for it, in a strictly engineering sense, may be. The through structure principle should be made use of on to the anchorages, with the necessary expansion break at the cantilever end, of course, and should terminate in grand portals, surmounting and dignifying the otherwise ungraceful and characterless anchorages. There would also be a utilitarian advantage in such treatment in securing greater immunity from fire dangers from adjacent buildings, and if the space beneath were to be utilized for fireproof structures, the additional headroom so secured would be valuable. This treatment would allow of a magnificent promenade, also a source of revenue.

The idea of adorning bridges is in itself not a new thing; in fact

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there are many examples of the attempts to create beauty by adding ornaments and so-called architectural devices, most of which, by the way, are complete and dismal failures. One of the most conspicuous cases the writer recalls is the disguising of a straight truss as a colonnade with false column bases and capitals and connecting arches of thin cast iron. The bridge is a double-deck structure, and the colonnade is supposed to have the effect of supporting the upper deck, regardless of the fact that the bases of the pseudo columns are resting on air. The effect is, of course, worse than grotesque—it is abominable. Mr. Crowell.

Sometimes an arch bridge of notable span length is utterly devoid of impressiveness and beauty because of the introduction of straight-line connecting spans between the ends of the arch and *terra firma*. Sometimes the effect of an otherwise magnificent arch is completely lost, because it is one of a pair instead of a single one, or one of three.

Ruskin says that the greatest art which the world has produced is that which is fitted for a place and subordinated to a purpose, and that all art worthy of the name is "good craftsmanship and work of the fingers, joined with good emotion and work of the heart; that is, the energy of body and soul united." If there be room for the application of this idea in the grand art of engineering, of which the writer thinks there can be no doubt, it will behoove the designers of monumental structures, like the one under discussion, to bring to their aid all the resources of art to produce in the highest attainable degree perfection of form conjointly with mechanical excellence.

J. E. Greiner, M. Am. Soc. C. E.—The author has developed a mechanical device, the object of which is to transfer the strain from the main cable to the backstay, thereby avoiding a continuous cable from anchorage to anchorage. It is hardly probable that any one will deny that the arrangement illustrated is novel, although socketed cables have been proposed before, the most prominent design being in the first-prize plan of the Budapest bridge. Mr. Greiner.

The question arises, will the saving in cost of erection justify the use of such an expensive arrangement, expensive not so much on account of the labor and material used in its construction as on account of the large amount of unavailable material placed in the cables? In the case under discussion, the details do not develop the strength of the members connected, and until experiments demonstrate that a socketed joint can be made strong enough to break the cables, engineers generally will avoid its use. As the capacity of the proposed bridge depends upon the strength of its cables, it may be well to inquire what excess of material must be placed in the cables in order that the sockets may be strong enough for the loads.

The tests quoted by the author show that sockets will, on an average, develop from about 70.4% to 83.1% of the strength of the sample plow steel wires, or from 76.8% to 85.9% of sample special steel wires,

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Mr. Greiner. and that, while tests of wires gave an average strength of 172 588 lbs. for special steel and 226 504 lbs. for plow steel, a difference of 31.2% in favor of the latter, the sockets attached to the plow steel cables developed an increased strength over those attached to the special steel of but 20.3 to 26.9 per cent. This indicates that the strength of the socketed cables does not increase in the same ratio as the ultimate strength of the different kinds of sample wires tested, and that the ratio is unfavorable to the socket.

According to the author's calculations, the cables will weigh 57 663 760 lbs. The socketed joints can develop from 70.4% to 85.9% of the strength represented by this weight, consequently there is from 14.1% to 29.6% of the material in the cables practically unavailable. This means that in order to use the proposed sockets it will be necessary to waste from about 8 000 000 to 17 000 000 lbs. of cable worth 7 cents per pound, or from $560 000 to $1 190 000. This is but a small percentage of the total cost; nevertheless, when dealing with millions, a very small percentage represents a large amount of money. If this amount can be saved in the erection, there can remain no objection to the excessive material used, and if the time of erection is lessened, the saving of interest on the large capital invested may result in a decided saving in the total cost of the completed structure.

It will be noticed that the author avoids expansion sockets over top of towers and allows the towers to deflect from the perpendicular. Similar means of taking care of expansion are in common use on a small scale, notably in train sheds. The application of the same principle on such a large scale is perfectly legitimate and appears to the writer as the proper thing to do in all cases of long-span superstructures. It is economical and perfectly practicable.

Other details of the design, namely, the anchorage and accessibility of all parts to inspection, are specially good features. In the writer's judgment, however, it would be best to cover the cables with close-woven wire thickly coated with white lead so as to protect them absolutely against the entrance of moisture. After being thus protected they could be covered by a layer of non-conducting substance which would allow heat from the sun's rays to reach the metal of the cables no faster than it is dissipated through the whole volume of the cables. In an address before the American Association for the Advancement of Science, read by Gustav Lindenthal, M. Am. Soc. C. E., on the "Economic Condition of Long-Span Bridges," it was proposed to use cables of parallel wires, but instead of being wrapped with wire, they would be enclosed in a water-tight steel envelope allowing an air space of about 2 ins. to prevent uneven heating of the wires by the sun. By using some such casing and packing the space with a non-conducting material, the wires could be kept at a nearly uniform temperature.

It has always appeared to the writer that a suspension bridge

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designed with stiffening trusses hanging to the main cables has something lacking in its æsthetic as well as mechanical construction. While a cable suspended from tower to tower assumes a beautiful curve and lends an appearance of grace to the bridge in spite of a crude and clumsy suspended stiffening truss, the fact should not be lost sight of that it is the curve which the cable assumes and not the comparatively thin cable which appeals to the æsthetic sense. When analyzing the beauty of a suspension bridge an appearance of bird-cage frailty will be found invariably in that part of the structure between the cable and stiffening truss, a lack of something not found wanting in an arch bridge. This void between the cable and the floor can be filled admirably by placing one cable over the other and bracing them together as proposed by Mr. Lindenthal in his design for the North River bridge. When arranged in this manner the cables will assume the same graceful curve, and will have the appearance of strength and rigidity where it is natural to expect it. If the cables of a suspension bridge can be utilized for chords of stiffening trusses, Mr. Greiner.

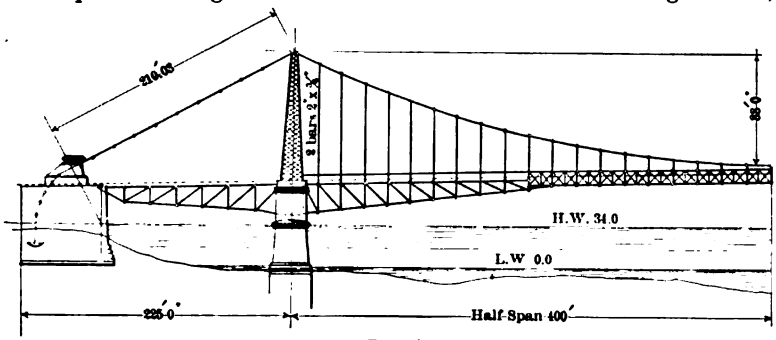

FIG. 26.

such an arrangement must naturally appeal to engineering instinct as the proper thing to provide, to say nothing of increased and substantial beauty and balance of the structure as a whole.

GUSTAVE KAUFMAN, M. Am. Soc. C. E.—The use of a cantilever in connection with the stiffening truss of a suspension bridge, as shown in Fig. 26, was contemplated by the writer several years ago. This plan was submitted by him to the authorities of the City of Pittsburg as a method for strengthening the Point Bridge at the mouth of the Monongahela River. In the event of its adoption by the city, some question might arise in the future as to its originality; therefore, to obviate any discussion on this point, its main features are shown here. The cantilever in this design is intended to relieve the cable of a large portion of its load; and it has been the writer's intention, since the conception of the design, to anchor the stiffening truss securely to the end of the cantilever, thus practically doing the same thing that has been proposed by the author. Mr. Kaufman.

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Mr. Meem. J. C. MEEM, Assoc. M. Am. Soc. C. E.—One of the new and interesting features suggested by the author, that of substituting cables and backstays, socketed at the towers, for continuous cables such as have been used heretofore in practice, possesses undoubted advantages, and the question then arises as to whether or not there are disadvantages sufficient to offset these advantages. In this method of construction the fundamental consideration is the strength of the socket; for, if a socket can be constructed which shall be, under all conditions, stronger than the rope itself, then necessity for further discussion is eliminated. The author suggests, however, that the strength be calculated for the strength of the socket rather than that of the cable, and this would necessitate, in a weaker socket, such as noticed in the tests given in the paper, a greater weight of cable, assuming that the strength of cable and socket vary in the same relation. The sockets used in the tests were of the ordinary wedge type and broke always just inside, beginning with the outer wires; and this, of course, showed unequal pull in the wires, causing the wedges to pinch or shear them off. The author suggested that a more satisfactory socket could be, and in fact had been, made. One such was described by Mr. Spilsbury, which depended in part for its strength upon the fact that the outer wires were consecutively bent and run in behind the butts of the succeeding wedges which bore upon them. As the author bases one of his objections to the continuous cables on the fact that the wires bend in passing over the saddle, and as this objection to the bent wires is also raised against the ordinary form of socket, it would also apply to the new form.

The conclusion is, then, that it is necessary to depend on a socket of this kind, which has its objections, or else upon one of the wedge type which is weaker than the rope. When it is considered further that a socket is dependent upon stress to develop its full strength, and that any variation, however infinitesimal, in this stress, tends in this infinitesimal degree to impair its strength, and that a socket under the same conditions would doubtless be more liable to injury from atmospheric changes, and when to these two weakening factors of vibration and wear are added the others of bending and shearing in the socket, the conclusion is that this is the weak point in the cable. If, by substituting continuous cables, the proportional number of sockets can be reduced from six to two, it would seem best to do this, even if at a somewhat greater initial expense of time and labor.

Strength might be added to the continuous cable in its passage over the saddle, by enveloping it in a casing of iron or steel, between the interior of which and the cable were ball bearings or rollers clasping it firmly, yet admitting of longitudinal motion. This would, perhaps, develop most fully the strength of each wire or rope by preventing any tendency whatsoever towards their displacement.

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T. KENNARD THOMSON, M. Am. Soc. C. E.—The author advocates using nickel steel, a metal supposed to undergo a permanent contraction at a temperature a few degrees below zero. As this change is sudden and must necessarily affect some parts of the bridge much sooner than others, it might be dangerous to use this metal in a bridge near New York. In a structure of this kind, where there is very little shock, it is a question whether it would not be safe to strain ordinary structural steel up to 20 000 lbs. per square inch. The writer was once ordered to design some movable short-span structures subject to very severe shocks, using a fiber strain of 20 000 lbs. per square inch. He refused absolutely to do this, and was allowed to go ahead in his own manner. He found afterward, however, that his structures were loaded to double the capacity used as a basis for the design, and many of them are still in service. Mr. Thomson.

GEORGE S. MORISON, Past-President Am. Soc. C. E.—Before entering upon more specific subjects, the author would call attention to the fact that the design which is made the basis of this paper is, like the paper, a study rather than an adopted plan. He considered that in a work of this magnitude and novelty it was necessary, not only to prepare a general design, but to work out all the important details to an extent which would show that there were no insuperable obstacles in the way of their construction, and no defects which might materially increase the quantities or cost of the structure. He did not consider that such design was a final design, either in its proportions or dimensions; while he is confident that it would produce a satisfactory structure, the cost of which would not exceed the estimates, he does not claim that it is perfect, but believes that many changes would be made if the construction of such a work were actually intrusted to him. Mr. Morison.

The claim that a suspension bridge with cables in parallel planes is as well fitted to resist wind pressure as one in which the cables are cradled is strangely at variance with the principles which have generally governed suspension bridge construction. While it may be true that under extreme conditions the resistance offered to wind by a bridge with parallel cables will be about as great as that offered by a bridge with cradled cables, it must be remembered that the disturbances which give a bridge a bad name, which shorten its life, and generally are most objectionable, are not the maximum distortions which occur only at long intervals, but the minute and frequent oscillations which, if they occur at all, occur constantly. It was these oscillations which proved the greatest defect in the old iron bridges, and which accompanied, as they often were, by small rattling members, gave an appearance of insecurity which was too often well founded. Though the cradled cables may have no advantage to resist a hurricane which would blow the bridge 8 ft. out of line (a storm which would work

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Mr. Morison. widespread havoc in the neighboring country and which has probably never extended over a few hundred feet at the same instant), they would have a very decided advantage in checking the oscillations, measured by fractions of an inch, which are caused by the passage of trains and due to other circumstances occurring every few minutes. Where the cables are in parallel vertical planes they, with the floor of the bridge, make a series of pendulums whose natural vibrations are perfectly synchronous and would readily become cumulative. On the other hand when the cables are cradled, each cable has a different motion, one goes up while the other goes down, and the motion of nearly every part of the structure greatly exceeds that of the center of gravity. The motions become complicated in so many different directions that their general effects neutralize each other, and the danger of cumulative synchronous effects, if not entirely eliminated, is very greatly reduced. A very simple experiment illustrates this difference. Suspend a horizontal bar by two parallel cords from a fixed bar above, set it vibrating in the direction of its length and observe the time which it takes for the vibrations to cease; then spread the cords apart at the top, so that their positions correspond to the planes of cradled cables, set it vibrating again, and observe the time it takes for vibrations of equal amplitude to cease. The time decreases rapidly with an increase of cradling.

If a stiffening truss of excessive depth is to be used, the central hinge is a matter of necessity; in no other way can the deflections of the cables due to strain and temperature be taken up without unreasonable stresses in the chords of the stiffening truss. Stiffening trusses with central hinges are described, and mathematical demonstrations of the strains in them given, in nearly all the text books which have treated of suspension bridges. They have seldom been built. There are two very serious objections to the central hinge. It is at best a loose detail in the point where stiffness is most needed, giving an opportunity for lost motion at the very point where the tendency to oscillation is greatest. The amount of motion on the central hinge is considerable, amounting in this case to something like 1° 20′, and if the stiffening truss is to be sustained by the cables throughout its length, the same bending motion must be concentrated at or near the central point of each cable; such concentration of bending motions would prevent the use of cables of large dimensions and perhaps would require a hinge in the cables as well as in the stiffening truss. A hinge which will permit of the necessary amount of motion and be adequate to carry the strains, both tensile and compressive, which will exist in a structure of this magnitude is believed to present mechanical difficulties which are practically insuperable. To obviate this difficulty it is proposed to make a hinge capable of resisting neither tension nor compression, using ropes or flexible plates in approximately vertical posi-

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tions. A hinge, if that is a proper name to apply to such a device, can be made in this way which will transfer vertical reactions, but it will be capable of no other duties. The whole value of the chords of the stiffening truss as members of a wind truss is thrown away, the chords being cut at the point where the wind strains are greatest. The wind must be resisted entirely by the cables; and cables, placed in vertical planes, can resist the wind only by swinging out of line until the suspended superstructure has been lifted to a point where its weight balances the wind. This will require a lateral deflection of 20 ft., instead of 8 ft., which seems fatal to this design. Mr. Morison.

The application of mathematical demonstration to test the relative merits of complicated designs before the details of those designs have been worked out is perhaps the most delusive abuse to which mathematical demonstration can be applied. It is believed that when the actual details of construction of the plan proposed by Mr. Mayer are developed with the same care that has been done in this paper, the plan will be modified in some very essential features.

The versed sine of one-eighth the span was selected largely because this ratio was used in the reports made by the two Boards of Engineers which have considered the subject of suspension bridges of this magnitude. It was not selected because the author believed it to be the most economical ratio. It was the natural ratio to select for a first study; but, before adopting any final design for construction, other ratios should be studied with equal care. The author believes that a less versed sine would be both more economical and in other ways better, but this would not affect the general details of construction, nor modify the cost enough to affect the general feasibility of such a bridge.

In like manner a deflection of 3.5 ft. was adopted as the permissible deflection in a length of 1 400 ft., because it corresponded to a 1% grade at each end of the curve. In view of the fact that the maximum deflection would occur only under a condition of loading which, though possible, is very improbable, and as trains of sufficient length to produce this deflection would balance themselves, one portion descending while another portion is ascending, the author believes that a deflection of twice this amount, or 7 ft., would be entirely unobjectionable. He would be prepared to recommend a permissible deflection of 5.25 ft., corresponding to a grade of 1.5% at each end of the vertical curve.

A reduction of the versed sine from one-eighth to one-tenth would, roughly, increase the weight of the cables 25 per cent. It would increase the portion of excess load distributed by the cables from 9.424% of the dead load to 11.78%; while it may be roughly stated that an increase of the permitted deflection from 3.5 ft. to 5.25 ft. would further increase the amount of excess load one-half, so that the portion of the

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Mr. Morison excess load absorbed by the cables would be 17.67% of the dead load; as the dead load is 39 000 lbs., this amounts to 6 891 lbs., which, deducted from 12 000 lbs., leaves 5 109 lbs. to be distributed by the stiffening truss, as against 8 325 in the design described in the paper. The depth of the stiffening truss, which is determined by the permitted deflection, would be reduced from 66 to 44 ft., and the theoretical section of the chords from 450 to 418 sq. ins. This means, in a general way, an increase of 25% in the weight of the cables, a decrease of 40% in the webs of the stiffening trusses, and a decrease of 7% in the chords of the stiffening trusses.

If the versed sine were reduced to one-twelfth and the permissible deflection increased to 7 ft., the effect would be, roughly, to increase the percentage of excess load distributed by cable to 28%, or 10 920 lbs. per lineal foot, leaving only 1 080 lbs. to be distributed by the stiffening truss, while the depth of the stiffening truss would be reduced to 33 ft. Such a design would probably be adopted for a highway bridge of these dimensions.

These illustrations are given as showing the modifications which are admissible in a design like that described in the paper, and which would be carefully considered before any such design is actually constructed.

It has been said that if distortions do not occur, the force which would produce the distortions must be absorbed somewhere in the structure. With trains running at high speeds, it is probable that the deflections would be reduced. The inertia of matter, which has comparatively little to do in small structures, would be important in a suspension bridge like the one described. The force of gravity is constant, but time is essential to the operation even of gravity. If gravity can be applied to only one-tenth the mass it has to move (as in the familiar experiment in which a weight of 4½ lbs. is placed at one end of rope passing over a pulley and 5½ lbs. at the other) it will take ten times as long for gravity to move this mass a given distance as it would to move the one-tenth to which it is actually applied. The same rule will apply to a bridge of this kind, with a limited time in which to create deflections. The deflections from light loads passing at high speeds will be less than from the same loads in a static position ; this means that trains which run at high speeds may have comparatively little effects on the great structure.

One of the principal features of the design is the use of wire ropes instead of independent wires to make the cables. These ropes would perhaps more properly be called strands. They are not twisted ropes of the ordinary character, but resemble the strands of which an ordinary rope is made up. Such strands are now made for various commercial purposes. They were selected some years ago by General Serrell as the best material to use for this purpose in his proposed

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bridge across the Hudson River at Fort Montgomery. The author believes that strands could be made much better adapted to this purpose than any which have yet been manufactured. In his first studies he contemplated making these strands of straight wire, but abandoned the idea because he did not believe that it was possible to transport them. A straight wire strand can only be handled when under a very considerable strain, and in strands of this length this strain can be maintained only by using the weight of the strand to produce it. Before actually manufacturing strands for use, experiments should be made to determine how small a twist is needed to insure the stability of the strand during transportation, and the author fully believes that this twist can be reduced to such a degree that the difference in strength between the twisted strands and the straight wire will be exceedingly small. Mr. Morison.

The experience of the East River Bridge is cited to show the necessity of adjustment of the different strands at the time they are placed. The author admits that such adjustment may be required under the sockets, but he believes that sufficiently accurate work can be done to render it unnecessary. He remembers the time when it was considered necessary to put adjustments in all the tension members of the webs of otherwise pin-connected trusses. A certain amount of error always exists and is permissible in every structure; the main cable is about 40 000 ins. long, and an error of 1 in. in the length of this cable would correspond to a variation of less than 700 lbs. per square inch in the strain on that cable; a variation of several times this amount would be taken up by the friction of the ropes upon each other. The author, however, would never think of using ropes in this way until those ropes had been actually subjected to a strain at least 10 000 lbs. greater per square inch than anything they would be expected to bear in the bridge, and had been kept under this strain for at least two days. This would be necessary to make sure of the uniform elongations which the tests made at Watertown show can be obtained.

To accomplish this he had contemplated building two substantial masonry piers at proper distances apart at the works where the ropes are manufactured, the space between these piers to be covered with a light shed, to keep out both rain and sun. On these masonry piers saddles should be securely fixed with bearings for the sockets, while suitable machinery should be provided for stretching the ropes under strain between these piers. The actual strains in the strands would be determined by measuring the deflection. Every rope, before it is shipped to the bridge, would be placed on these piers, adjusted under strain to the proper length, the sockets fitted on and trimmed by a special tool which would turn the bearing after the socket is on the rope, so that the deflection would correspond to the exact strain de-

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Mr. Morison. sired in the rope. These piers should be made of such dimensions that twenty or more ropes could be placed there at the same time, and the deflections of the ropes first placed would be used to measure the deflections of the later ones. No rope should remain in this testing shed less than two days. After testing, each rope should be rolled up in a coil of considerable diameter (perhaps too large to transport on a railroad car) so that the compacting effect of the strain would not be modified by the coiling of the rope. The author fully recognizes the fact that the elongation which would occur in a twisted rope which had not been subjected to strain in this manner might be fatal to anything like uniformity in the action of the cables. By this method he believes that these difficulties can be removed.

The ropes used as suspenders, though of the same weight and sections as those used in the main cable, are subjected to less than half the same strain per square inch; and as they pass around saddles of about 40 ins. diameter, it would probably be expedient to use softer wire in them than in the main cables.

The sockets at the ends of the cables may be elements of weakness, but they do not necessarily mean more than a screw at the end of a rod or the heads at each end of an eye-bar. The same kind of objections that are now raised to a socket on a wire rope were raised thirty years ago to an upset head on an eye-bar, when some of the best engineers absolutely refused to allow eye-bars to be forged by a process which so weakened the fiber of the iron, and welds in the body of the bars were preferred by many to any upset enlargement for a screw. It is only within the last ten years that there has been anything like a reasonable certainty of securing eye-bars in which the head would break the body of the bar, but engineers accepted upset bars as better and safer than any welded bars that could be made, recognizing that while the heads were not what they would have been glad to have, the fractures, which they could not avoid, occurred under strains far above any working strains and in no way affected the elastic limit.

The change which is now going on in the manufacture of sockets is of like kind. Mr. Spilsbury states positively that he can make a socket which will develop the full strength of the rope; even if this were not so, the strength of the rope in the socket can undoubtedly be increased to a limit which will compare favorably with the usual strength of eye-bar heads, and which will be so far in excess of any working strains that the weakness is more a matter of theory than of practice. From its very nature the weakness of a socket is developed principally when the contraction of the wires under extreme strain prevents their filling the full area of the socket, and in hard steel wires this contraction occurs only at strains which are relatively far in excess of the elastic limit in steel and iron bars.

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Mr. Morison.

Wire ropes are used in many places where the strength of a socket is absolutely essential to safety. A single rope used for a carrier in a wire tram is dependent on two sockets, the failure of either of which would produce disaster. In the cables of the proposed bridge there are 253 ropes, and the risk is divided among a corresponding number of sockets, the failure of several of which might occur with no serious result.

There is no question but what the detail adopted for the connections at the top of the tower is heavy and expensive. Practically the entire weight of these special connections would be saved if the cables were made continuous from anchorage to anchorage. Furthermore, if the side spans were suspended from the backstay cables, those cables could be made to terminate horizontally in the anchorage, thus reducing the strains on the socketed connections more than 10%, when the versed sine is one-eighth, below the maximum strains in the cables. There were, however, several reasons which made the author prefer to use the detail described in the paper.

A tower which is unsupported laterally at the top requires a large base to secure sufficient stability; on the other hand, a tower which is held at the top becomes simply a post, like the guyed mast of a derrick, which may be made of much smaller dimensions. The only way to guy such a slender tower satisfactorily is to fasten the cables rigidly to the top. The detail adopted holds the top of the tower rigidly and further makes equal inclinations of main cables and backstays unnecessary. Where there is no special reason for doing otherwise, equal inclinations would probably be selected, but where land is valuable the cost of a site for an anchorage might be greatly reduced by increasing or decreasing the inclination of the backstays. The friction of cables passing over a curved saddle would probably be enough to hold the top of the tower securely, but engineers hesitate to depend on friction in places where a slight movement against friction would produce an injury which could not be repaired. In spans of moderate length where the two inclinations are equal, continuous cables might be the best practice. In such spans as the one considered, the author would hesitate to use them unless the towers were made of such dimensions as to depend entirely on the base for their stability, and this involves movable saddles.

The plan designed simply suspends independent ropes between two points. The strains in these ropes can be determined by their deflections, thus insuring practically uniform strains in the several ropes which compose the cable. If the ropes were to be run continuously from anchorage to anchorage, they would be suspended at four points, and their free motion would be affected by the friction on the bearings over the towers. It would be possible to adjust them to very nearly uniform strains, but the time required for such

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Mr. Morison. adjustment would be considerable, and the danger that an occasional rope, supported for its whole length on the ropes below, would be slack without the slackness being observed, would be very greatly increased.

It is absolutely necessary that every rope should be strained, as already explained, before it is put in the bridge. The difficulty of subjecting ropes to such strains increases rapidly with the length of the ropes. A deflection of 66.5 ft. corresponds to a strain of 70 000 lbs. per square inch of a rope 3 342 ft. long, the length of the main cable; whereas, the deflection corresponding to the same strain in a rope 5 878 ft. long, the total length from anchorage to anchorage, is 205.7 ft. The towers and sheds required for the former test would be comparatively simple structures of practical dimensions. The towers and sheds required for the latter test would be of prohibitory size. It would be necessary to resort to the complications of intermediate supports with the uncertainties which would attend their use.

The connection between the two sets of cables at the top of the tower would have to be very carefully made, and involves some difficulties of construction. The author at first contemplated using single pins in place of the 5-in. bolts, and discarded them simply because of the large dimensions which these pins made necessary. The vertical members which hold down the lower ties could be slackened after the upper ties are in position, so that the strains would be divided between the two sets of ties as designed, by the principles of leverage, in an inverse ratio to the distances of the ties above and below the line connecting the theoretical intersection points. The duty of the several 5-in. bolts corresponds very largely to the duties of rivets in riveted joints. Accurate work would be required, but accurate work can now be had, and with the present facilities for electric heating of local members, connections can be made which a few years ago were impossible. The long pins or bolts could not be driven by the battering ram usually used in bridge erection, but must be forced in slowly by hydraulic or other extreme pressure. While the author admits that the construction of these connections is a nice piece of work, he has no doubt of the ability to carry it out in a manner which would produce satisfactory results.

The unequal length which the splaying out of the ropes to form the connections will make in the length of the separate ropes can easily be provided for by varying the depths of the holes in washer plates into which the sockets fit. The point raised by Mr. Lindenthal, that the strains in the outer ropes would be seriously increased by the cradling of the cables if these are first strung in vertical planes, is well taken. The author confesses that the calculation of the strains which would be produced in this manner has given results which he

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had not expected. The plans described in the paper should be modified to the extent of laying the cables originally in their cradled position, at least through the splayed portion, while it would probably be expedient to continue the cradling a considerable distance further. This could be accomplished by connecting together the iron horses in which the opposite cables are laid. It would also be expedient to stop the wrapping 100 or 150 ft. from the top of the tower instead of 50 ft., as shown in the design, enclosing the intermediate portion of the cable in a steel envelope somewhat similar to that which Mr. Lindenthal has proposed to use for his North River Bridge. By this change the increase of strain in the upper ropes caused by the deflection of the cables would be very much reduced. The extreme estimated deflection below the normal position at the center of the span is calculated as 7.11 ft., which corresponds to 0.46 ft. at the end of a tangent 50 ft. long, and to 1.88 ft. at the end of a tangent 150 ft. long. The former corresponds approximately to a variation of 10 500 lbs. per square inch in strain on the outside ropes, and the latter to a variation of 4 200 lbs., which, occurring only in very warm weather, is well within permissible limits. Mr. Morison.

The author recognizes that the deflections of the towers during construction would be considerable, and that they would be much greater if no weight is carried by the backstays than if the backstays support the side spans. It would, of course, be possible to take up this deflection by bending the tower, additional material being provided to resist such bending; but the author does not believe that a careful estimate would show this method to be economical, and it would probably involve using a more slender tower than has been designed, the base having been fixed in a measure by the necessary dimensions of the foundations, including proper space to work between the several independent cylinders. In a small bridge, in which the number of foundations could be reduced, this method would readily be adopted, or the tower could be hinged at the bottom. The author has had in mind a somewhat original method of getting over this difficulty, and, that is, to support the posts of the tower on the river side on cylinders filled with lead, which lead would be under a pressure far in excess of that under which lead flows. By tapping holes in these cylinders the lead would be allowed to flow out and the posts would settle to their true position, the flow of the lead being so slow that the settlement could easily be checked or regulated. In other words, the posts would rest temporarily on hydraulic presses, in which the liquid used is lead.

As regards the connection at the bottom of the anchorage, where no rock exists in which to place the auxiliary anchorage for use during construction, it would either be necessary to provide additional masonry or make some other temporary device by which the plates would be

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Mr. Morison. held in position. The author does not regard the necessary transmission of a considerable pressure through green masonry as a serious thing. It would once have been so, but a mortar made with good Portland cement becomes hard in so short a time that no trouble need be apprehended from this source. On one occasion he constructed a number of small brick piers, with steel rods passing through those piers from base to top, and instructed the resident engineer in charge to screw up these rods until there was a strain on them of 15 000 lbs. to the inch. The resident engineer, expecting compression in the piers, built them slightly higher than the plans called for, and on their completion immediately screwed up the rods until the elongation shown by the screw threads indicated that the necessary strain was developed. The piers continue to this day as much higher than the plan as he had built them.

No final specification has been drawn for the steel of a bridge like the one described in the paper. The author thought it best to provide for nickel steel for the portions of the chords which are subjected to the greatest reversal of strains; one of his reasons for doing so was the objections which he knew would be raised to the use of ordinary steel under these conditions. So far as the author knows, experiments made on high carbon steel, under conditions in which strains are frequently reversed, show an ability to resist such reverse strains about equal to the ability of nickel steel having the same elastic limit. He believes that this bridge would be perfectly safe for the service which it has been supposed to perform if high carbon steel without nickel were used in the chords. He would prefer, however, not to take the responsibility of using what is generally considered a brittle material when a tougher material of equal strength is available, though at greater cost. The case of the St. Louis Bridge is cited to show that a very hard steel which is sometimes subjected to tension as well as compression, has resisted unusually high strains for 22 years without showing any defect; if it could be proved that no fractures exist in this steel the author would consider this strong evidence in favor of the use of high steel under correspondingly high strains, but unfortunately no one knows the present condition of the steel in the arches of the St. Louis Bridge. Each section of arch is formed of six staves of hard, high carbon steel (not chrome steel as is commonly reported), enclosed in a riveted jacket of softer material; the staves are utterly inaccessible, and fractures may have occurred without being known.

The author recognizes the objection which may be made to a cross-bracing between the two stiffening trusses which is not sufficiently strong. The bracing used depends on the stiffness of the broad web members between the connection points and the bottom chords, a practice which is in general use in all through bridges. He would prefer to extend his diagonals to the bottom chord, but the arrange-

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ment shown will give almost equally good results, provided the web members have proper lateral stiffness. Mr. Morison.

The study gives the design of an engineering structure, a tool intended to do work, and not a monumental design for ornament or commemoration; undoubtedly some of the features might be modified in a way which would enhance beauty. The author thinks that such modification would have been out of place in a paper like the one under discussion, "submitted with a view to opening the way for improvement and to show that a great suspension bridge, which would be well adapted to railroad service, would involve no insurmountable difficulties of construction." He has, however, long felt that no object can be really beautiful in which adaptation to its purpose and correct construction are not the fundamental features of the design. Ornamentation may mar or beautify according to the skill with which it is done, but the artist who knows nothing of construction is no more competent to pass on the beauty of a great bridge than is the man without musical education to criticise the classical works of great composers. No decoration however bad, unless it conceals all constructive outlines, can destroy the beauty of a correctly designed and proportioned work, and no decoration however good can render beautiful a work built on poor lines of construction. Nature may abhor straight lines, but Nature has built with an excess of material which engineers cannot have, and yet in the needles of a pine forest, in the trunk of the palm tree and in the prisms of columnar basalt, it has shown that it has use for straight lines. A curved line is unfitted to resist either tension or compression unless accompanied by modifying transverse strains; a tower held at both top and bottom is simply a compression member which construction demands shall be straight and whose real beauty would be destroyed if curved lines were introduced. The design advocated by Mr. Mayer substitutes curves for the straight lines of the top chord of the stiffening truss selected by the author; if a comparison be made of the beauty of the two structures, the author has no fear of the result.

Two monuments, both bearing the name of Washington, have been built which illustrate the author's idea of beauty so well that they may be cited here. One stands in the city which also bears his name, a single upright shaft, perfectly plain, every line of which is straight; it was built by an engineer; it stands there with no other beauty than that given to it by its perfect proportions, one of the few of man's works which looks as if it might have had a divine origin, like a single index finger pointed to heaven. The other is in New York, in a square which also bears the name of Washington. It is an arch in which all the principles of monumental construction are violated by making the abutments so thin that the first thought of the observer is of the effort they must make to resist the thrust of the arch. Where perpetual rest

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Mr. Morison. should prevail, we find the torture of eternal strain; exquisite decoration lavishly applied fails to cure the false proportions, and the curved arch of the architect is as ugly as the straight obelisk of the engineer is beautiful.

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EXPERIMENTS ON THE PROTECTION OF STEEL AND ALUMINUM EXPOSED TO SEA WATER.

By A. H. SABIN, Assoc. M. Am. Soc. C. E.

PRESENTED NOVEMBER 4TH, 1896.

WITH DISCUSSION.

In February, 1895, the author, by permission of the Navy Department, suspended a cage containing twenty iron plates coated with various preservative coatings in the sea water in the Brooklyn Navy Yard. At the expiration of six months it was found that by some accident the cage and all the plates had been lost. Arrangements were then made for another and more carefully conducted test, and in making these arrangements the author was assisted by Naval Constructor F. T. Bowles, to whom he takes this opportunity of expressing his appreciation and thanks. Without his co-operation it would have been impossible to carry out the experiments. The work of preparing and applying the coatings was done by Edward Smith & Company, of New York City, who paid all the expenses incident to the whole work, and the aluminum plates were furnished free of charge by the Pittsburgh Reduction Company.

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It was desired especially to learn the protective capacity of various varnishes and of what are known as enamel paints, which are pigments ground in varnish.

The ordinary oil and resin varnishes are made by compounding certain resins (of which those selected for these experiments are Kauri, Zanzibar and Manilla, and the resinous mineral known as gilsonite) with various proportions of refined linseed oil, and these oil and resin compounds are then thinned with spirit of turpentine, which is commonly regarded as a vehicle, so that the essential differences in varnishes are due chiefly to the kinds of resin used and the proportion of oil. For convenience, it is customary to assume 100 lbs. of resin as the unit, and with this compound 8 galls. of oil for one kind of varnish, or 12 galls. for another, or 20 galls. for another, or 30 galls. for another; and it was believed that for the purpose of these experiments these four types of varnishes, commonly spoken of as 8, 12, 20 and 30-gall. varnishes, were sufficient. It was also regarded as desirable to know what resins were most suitable; hence there were prepared such series of varnishes from Kauri, Zanzibar and Manilla resins, and an 8-gall. Kauri varnish is designated as 8 K., an 8-gall. Zanzibar as 8 Z., an 8-gall. Manilla as 8 M., and so on.

To make what are known as enamel paints, certain pigments are ground in these varnishes, and it was thought desirable to ascertain what effect the character of the pigment has on the durability of the paint. Hence, in some cases, flake white, which is pure white lead, was used; in others, white zinc; and in others, pigments thought to be chemically inert, as ultramarine blue and chromium oxide, and also a very permanent red pigment known as flamingo red, the exact composition of which is not known.

Inasmuch as it has been believed that the process of baking adds to the durability of these coatings, in most cases duplicates were prepared, one of which was baked and the other allowed to dry at the ordinary temperature.

Besides these varnishes and enamel paints, plates were coated with ordinary red lead in oil, with two kinds of iron oxide paints in oil, with an iron oxide paint mixed in a shellac compound after a formula furnished by Naval Constructor Bowles, with ivory black in japan, with Edward Smith & Company's "Durable Metal Coating," which is a varnish in which a considerable amount of gilsonite has been sub-

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stituted for resin, and with the "Sabin Process" enamel, which has been used on some of the most important hydraulic work done within the past few years. It is hoped that a careful study of the effect of sea water on these various coatings, part of which were applied to steel and part to aluminum plates, will lead to important advances in the knowledge of the subject.

The steel plates used were ordinary boiler steel about $\frac{3}{16}$ in. thick, purchased from a well-known firm of boiler makers. In all cases where two steel plates were coated with the same preparation, the one bearing the odd number was dried at the ordinary temperature and the one bearing the even number was baked at a temperature of 215° to 240° Fahr. for four hours or longer; except that all the "Sabin Process" pipe coating enamels were baked two hours at 400° to 450° Fahr. Each plate was stamped with a number before coating. The following table gives the coating applied:

STEEL PLATES.

No.	Coating.
1 & 2	Flake white in 20 K.
3 & 4	Flake white and white zinc, equal parts in 20 K.
5 & 6	White zinc in 30 K.
7 & 8	White zinc in 20 K.
9 & 10	" " in 12 K.
11 & 12	" " in 8 K.
13 & 14	" " in 30 M.
15 & 16	" " in 20 M.
17 & 18	Flake white in 8 M.
19 & 20	White zinc in 30 Z.
21 & 22	" " in 20 Z.
23 & 24	" " in 8 Z.
25 & 26	Flamingo red in 20 K.
27 & 28	Ultramarine blue in 20 K.
29 & 30	Chromium oxide in 20 K.
31 & 32	Durable metal coating.
33 & 34	" " "
35, 36, 37 & 38	"Sabin Process" pipe coating enamel.
39	Ivory black in japan.
40	Red lead in oil.

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41.....Prince's metallic paint (oxide of iron) in oil.
42 .....Purple oxide of iron in oil.
43 & 44 .....No pigment, 8 K. varnish.
45 & 46.....No pigment, 12 K. varnish.
47 & 48..... " " spar varnish.
49 & 50..... " " 30 K. varnish.
51 & 52..... " " wearing body varnish.
53.... .....Iron oxide in shellac mixture.
54.....Parahydric coating.

When these plates were removed from the water on July 29th, 1896, after six months' immersion, they were immediately inspected by Naval Constructor Bowles, Foster Crowell, M. Am. Soc. C. E.; William Barclay Parsons, M. Am. Soc. C. E.; Mr. S. V. V. Huntington, Manager of Edward Smith & Co.; James C. McGuire, Jun. Am. Soc. C. E., and the author. The plates were afterward removed to the author's laboratory, and, after an interval of a month, were carefully examined, with the following results:

1. Good; first-rate condition.

2. Good, but some small marginal blisters. No corrosion.

3 & 4. Like 1 and 2. Coating could be peeled off with a knife when first taken from the water.

5. Very thin near the edges; some marginal blisters.

6. Same; coating brittle where very thin, elsewhere tough; no corrosion.

7. Like 5. Fifteen or twenty small rust spots where the coating was very thin. Brittle.

8. Good condition; no blisters, but coating brittle; not easily scraped off when wet.

9. Hard and brittle, but not completely removed by scraping while wet. A large number of pin-hole rust spots on one side.

10. Good; no blisters, but hard and brittle, especially where thin.

11. Hard and brittle; no blisters; not easily removed by scraping while wet. The outer coat separated from the under coat when scraped, leaving the latter on the metal.

12. Some small blisters and rust spots on one side. Hard and brittle.

13. Hard and brittle, very many minute rust spots. General condition poor.

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14. Could be completely scraped off with a knife when wet. Very little rust. Both 13 and 14 were tough and good where the coating was very heavy, brittle where thin.

15. Thin. General condition good, except along the margins, where there was some rust. Brittle.

16. Coating thicker on one side than the other. The thicker film gave good protection, but the other side was brittle and showed numerous rust spots.

17. Coating badly decomposed, the action taking place from the outer surface. Not much corrosion.

18. Some marginal blisters, but general condition pretty good; much better than 17.

19. First-rate condition, tough and adherent; not completely scraped off with a knife when wet; rather thin along edges.

20. General condition perfect, but could be scraped off when wet.

21. Good condition, but thin and brittle near edges; not easily completely removed.

22. Same, but coating could be scraped off when wet.

23 & 24. Thin, brittle, many rust spots; poor condition.

25 & 26. Many rust spots; general condition bad.

27. Not much rust, but many blisters; condition not good.

28. Better than 27, but very many small blisters.

29 & 30. About alike; very many small rust spots.

31 & 33. Some blisters along the margins where the coating was thin; elsewhere the coating was all right; could be scraped off when wet.

32 & 34. Same as 31 and 33, except that there was much more blistering along the edges. Not good.

39. Very bad; rusty all over.

40. A good many small rust spots, but no general corrosion. Coating considerably decomposed; could be scraped off completely, but not very easily. General condition fair.

All the pipe coating enamels were perfect.

Plates 41 to 54 were in a separate cage, which was destroyed by an accident after about three months' immersion, and all the plates were lost. This was very unfortunate, as these would have shown some well-known oil paints in comparison with the others, but the test will be repeated.

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Thirty aluminum plates, coated with various paints and enamels, were removed from the sea water in the Brooklyn Navy Yard, July 29th, 1896, after immersion for six months.

The composition of these plates was as follows:

Series I.—Ninety-nine and one-half per cent. pure aluminum.

Series II.—Ninety-eight per cent. aluminum and 2% copper.

Series III.—Ninety-eight per cent. aluminum (the quality known as commercially pure aluminum).

Series IV.—Ninety-three per cent. aluminum and 7% copper.

Series V.—Seventy-five per cent. aluminum, 20% zinc, 3% copper, 1% iron.

These plates were numbered from 101 to 130, and were coated as follows:

I.	II.	III.	IV.	V.	
101	107	113	119	125....	"Sabin Process" pipe coating enamel, baked.
102	108	114	120	126....	Durable metal coating, one side baked.
103	109	115	121	127....	Ultramarine blue, one side; flamingo red, one side; ground in varnish, not baked.
104	110	116	122	128....	White zinc ground in varnish, one side baked.
105	111	117	123	129....	Chromium oxide ground in varnish, one side baked.
106	112	118	124	130....	Edward Smith & Co.'s spar varnish, no pigment, one side baked.

The varnish in which the ultramarine blue, flamingo red, white zinc and chromium oxide were ground was composed of 100 lbs. Kauri resin to 20 galls. linseed oil, thinned with turpentine. The chromium oxide was the anhydrous oxide made by the ignition method, and was of commercial quality, not chemically pure. The baked coatings were baked about four hours at 215° to 240° Fahr., except that the "Sabin Process" pipe coating enamel was baked two hours at 400° Fahr.

The condition of each plate is given in the following statement:

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SERIES I.

101. Perfect.

102. Baked side, perfect. Unbaked side, three blisters ⅛ in. diameter. No general corrosion or roughening. The surface of the paint had lost its gloss. The coating was good on the edges of the plates.

103. Ultramarine blue. Showed roughening of coating, numerous pin head blisters, no corrosion to speak of.

Flamingo red. General condition good, except near edges of plate, which showed blisters over a surface about half an inch wide and one-fifth the marginal distance; very little corrosion.

104. Baked side. About 2 sq. ins. in one place half covered with small blisters. No corrosion.

Unbaked side. First-rate condition.

105. Baked side, one blister, 1 x ½ in.; otherwise first rate. No corrosion. Unbaked side all right.

106. Both sides perfect.

SERIES II.

107. Perfect.

108. Baked side, one blister, ⅛ in. diameter.

Unbaked side perfect.

109. Blue and red about the same as 103, except that about twice as much surface was blistered. General condition good.

110. Baked side badly blistered in spots along the edges, amounting to about 6% of the total surface of the plate. Some corrosion under these.

Unbaked side all right, except that about 1% of the surface showed pin-head blisters along a strip about ½ in. wide on one edge of the plate.

111. Baked side showed four central ⅛-in. blisters, numerous marginal ones about 1½% of plate. Very little corrosion.

Unbaked side in first rate condition.

112. Baked side, two central blisters 2 sq. ins. and 4 sq. ins., and nearly all the margin ½ in. wide. Considerable corrosion. Otherwise perfect protection and high luster.

Unbaked side, two central blisters ⅛ sq. in. and 1 sq. inch.

Slight marginal corrosion. Coating evidently thin on edges.

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SERIES III.

113. At one corner evidently a break in the coating let in water and caused a blister of about 2 sq. ins. Coating rather overbaked and brittle; elsewhere perfect.

114. Baked side perfect.

Unbaked side tough and adherent, except one small spot near the middle of the plate, which looked as if the coating had been broken, and where corrosion had begun.

115. Blue and red about alike. No decided blisters, but coating itself showed some signs of decomposition, especially the blue, which had a rough surface.

116. Both sides in good condition, but showed some signs of incipient blistering about the edges.

117. All right on both sides.

118. Both sides quite perfect.

SERIES IV.

119. At several places about the corners of the plate single blisters, some of which were as large as 3 sq. ins., had formed. These appeared to be due to the fact that the coating was overbaked and had been cracked at the corners by the supporting framework, and galvanic action had ensued on the penetration of the sea water. This was facilitated by the 7% of copper in the alloy. The remainder of the plate was perfect.

120. Baked side showed three blisters of about 1 sq. in. each and some corrosion under these; otherwise all right.

Unbaked side perfect.

121. Blue and red about alike; about 30% blistered and corroded.

122. Pin head blisters along the edges; general condition all right.

123. Baked side all right.

Unbaked side, seven or eight small blisters, but no corrosion. General condition good.

124. Both sides badly blistered and corroded along the edge, about 10% of the surface. Where not blistered, all right.

SERIES V.

125. Coating brittle and certainly overbaked. Badly blistered along the edges. In all cases of blisters under pipe coating enamel, the blisters were continuous and start from the edge.

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The middle of the plate was all right.

126. Baked side badly blistered along the edge, 6% or 8% affected.

Unbaked side slightly blistered, chiefly along one edge; condition otherwise good. No corrosion.

127. Blue. Considerably blistered along the edges, in pin head blisters mainly. Little corrosion.

Red. About the same, but some large marginal blisters. The red had a smooth surface, but the blue was rough.

128. Baked side, nine or ten blisters of some size (1½ ins. diam.), and considerable corrosion. Remainder of surface good.

Unbaked side. About 1% of the surface, near the edges, with small blisters showing some corrosion. The rest of the surface all right.

129. Baked side. A large number of groups (about 1 in. diam.) of small blisters. No corrosion.

Unbaked side. About the same, but not as bad.

130. About like 124.

The pigment mentioned as flamingo red is supposed to be a mixture containing some coloring matter derived from coal tar, and is reasonably permanent in the air. In these tests it became dark and mottled.

The baked surfaces of those plates which had one side baked were in all cases harder and more glossy than the other sides, after the test was made. It should be observed, however, that while these coatings are all at present hard and firm, when they were first taken from the water they were much softer and could be more easily scratched or scraped off, with the exception of the pipe-coating enamel, which was exactly as it was when it was put in the water. Even the luster was not affected, and the smooth plates coated with it are like a mirror.

On all the plates, except those coated with pipe-coating enamel which was applied by dipping, the coating is much thinner for about an inch along the edges of the plates than it is on the central portion. This fact has been called to the attention of the workmen who painted the plates, and is said to result from the method employed in applying the paint with a brush. In future experiments, care will be taken to avoid this. Probably four-fifths of the corrosion occurred along this marginal strip.

The more nearly pure the aluminum, the less it seems to need protection. The series which contain large amounts of copper or zinc are the most difficult to protect; corrosion seems to progress rapidly when

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the sea water gets the slightest admission, and it throws off the coating, no matter how impervious the latter may be.

These plates were suspended in an open oaken cage or rack, the plates being in a horizontal position, one above another, about 2 ins. apart. They fitted loosely in the rack, and each plate was supported at the four corners. The rack was suspended from a float, and was 6 ft. or so below the surface of the water.

In fitting these plates to the wooden frames, they were originally tight, but became loose as the water swelled the wood. In transporting them to the Navy Yard, putting them again in place, and the subsequent handling, it is quite possible that scratches may in some cases have been made in the coatings, which would account for some of the blisters on plates which were otherwise perfectly protected. Greater care will be given to these details in the future.

Other experiments indicate that there is probably considerable sewage in the water, as sulphides are present in appreciable quantity. Something in the water also discolored the white zinc used, but did not act except superficially.

White zinc appears to be the least acted on of any pigment used in this test. The durable metal coating gave excellent results. When first taken from the water, however, this coating was somewhat soft, though extremely tough and elastic, and could then have been scraped off. After drying, this was impossible.

The most satisfactory coating is the pipe-coating enamel, which was baked on; and it is probable that if it had not been for the accidental overbaking of some of the plates, a condition difficult to avoid in using a small oven though easy to regulate in a large one, all the plates thus coated would have been perfectly preserved. The other baked coatings did not do so well. It appears that an extremely high degree of elasticity is very essential in these tests.

In the foregoing notes, where it is said that there was "no corrosion," it is meant that there was no appreciable roughening of the metallic surface, which was bright when the paint was removed. Where corrosion is noted, it is meant that a deposit of oxide was found under the paint. In all cases of blistering there had been a little corrosion.

It is proposed to replace a considerable number of the plates, both steel and aluminum, for a further six months' test.

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The author has at present no explanation to make of the results so far presented, but makes one or two suggestions. Charles B. Dudley, M. Am. Soc. C. E., has a theory that varnish is so constituted as to have a part of the oil which it contains in chemical combination with the resinous material, and a part of the oil free. If this theory is correct, it would be reasonable to suppose that the longer the oil and resin are heated together in the process of varnish making, and the higher the temperature employed, the more complete will be the combination of oil and resin, and the less free oil will remain. Now, for certain reasons it is not practicable to heat 8-gall. and 12-gall. varnishes nearly so much as 20-gall. and 30-gall. varnishes; and if one action of sea water is to dissolve the uncombined oil, it is plain that the varnishes which contain the most oil, but are most thoroughly combined, will suffer less than those in which the proportion of oil is less. This would explain the singular fact that while it appears both by these tests and by other and more general experience that a suitable combination of oil and resin has greater protective action than oil alone, yet the smaller the proportion of resin, the more durable the compound. It may be objected that oxidized linseed oil is itself a very permanent substance, but, while this is so, it is also true that it is easily dissolved by alkalies, and even by water containing ammonia, and it is quite possible that it may be rather easily soluble in sea water. Further experiments are needed on this point.

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DISCUSSION.

Mr. Bowles. FRANCIS T. BOWLES, Esq.*—The speaker wishes to express his appreciation of the care taken by the author and his firm in carrying out the tests described in the paper. The greatest attention must be paid to the preparation of such plates in the first place, and to their subsequent immersion, care and inspection. For the last ten years the speaker has been in charge of docking, painting and cleaning the bottoms of ships, and during that time has made many experiments with plates. He found that conclusions drawn from experiments upon a series of plates did not agree with experiments with the same materials upon the bottoms of ships, although there was no apparent reason for this difference. Hence it is impossible to form definite, certain conclusions from any series of paint tests on plates, no matter how carefully they are made, even if they are distributed all over the coast in various kinds of water. The immersion of the plates at the Brooklyn Navy Yard is an indication of what would occur in other places. The water at the Navy Yard is very foul with sewage and the collections of the harbor, which fact serves to increase the apparent duration of the test by increasing the rapidity of the action of the paint.

It has been determined that good unpainted iron or steel plates will lose on one surface in 100 years from .30 to .50 in. thickness if immersed in sea water, .02 to .03 in. in pure, fresh water, and from .25 to .30 in. if exposed to the atmosphere. The problem of covering the bottoms of ships divides itself into two parts, the protection of the metal against corrosion, and the protection of the outer surface of this covering against fouling. The qualities of paint which prevent corrosion are apparently those making it water-tight. The paint must be elastic in order to provide for the changes of temperature and the strains to which ships are subjected when afloat. It must be strongly adherent to withstand the abrasion and usage ships receive in docking and alongside wharves. The qualities of paint which prevent fouling are those making it glassy, that is, a smooth, hard substance inert in air and water. Fouling has never been known to attach itself to clean glass, although if the latter be exposed to the action of water from which mud or slime will be deposited upon it, then marine growth attaches itself at once.

The subject of painting ships has attracted the attention of many people for a long period, one of the earliest patents, dating from about 1670, being for coal tar and asphalt varnish. The importance of the subject is shown by the fact that while the fouling of the battleship *Indiana* between her launching and trial was very slight, it was suffi-

* Naval Constructor, U. S. Navy.

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cient to make a difference to the builders of $100 000 in the premium earned for speed. It probably increased the resistance of the ship about 15 per cent. A painted vessel of ordinary size, after six months in the water, requires at least $1 000 for paint alone. A ship subject to ordinary service, staying in port about half the time will foul so rapidly that in from six to nine months there will be a loss of speed of from 25 to 40% ; where ships are running regularly and rapidly, the loss is less. Mr. Bowles.

The paints now used on bottoms of ships by the United States Navy Department are chiefly the McInnes, an English preparation, and the Rathjens, which is German, and these illustrate the two classes of paints generally employed for the purpose. The McInnes paint consists, first, of the so-called anti-corrosive paint, a pigment of iron oxide with a very quick drying vehicle and a small amount of oil. It dries very rapidly, a matter of commercial importance in reducing the time, and, hence, the fees of docking, On top of the anti-corrosive paint is applied copper soap, a thick, pasty, green substance, which dissolves slowly in sea water and forms salts poisonous to marine growths, thereby preventing their adherence to the surface. The Rathjens anti-corrosive paint is of the same character as that already mentioned, but the anti-fouling paint is a heavy, brown paint of which the pigment is largely oxide of iron. It contains a considerable amount of oil and certain insoluble poisons, which apparently choke marine growths, it being a peculiarity of soluble poisons that they will not affect the growth of barnacles.

Varnishes have seldom been used on the bottoms of ships, although various trials of them have been made. Their defects seem to be that their coefficient of expansion is not the same as that of metal, that they gradually disintegrate, and that water gets below them, destroying their adhesive power altogether, and causing them to come off in sheets and strips.

William Barclay Parsons, M. Am. Soc. C. E.—The packing of the plates in the crates and their subsequent handling undoubtedly injured the edges considerably more than the author has stated, as is apparent from an inspection of them. The ease with which many of the coatings could be removed when wet was noticeable. When taken from the water many were in such a condition that they could be removed in large strips by means of a sharp knife. When a coating was removed in this way, the metal below was bright in some cases, showing that, while the action of the sea water on the paint had been of a softening nature, there was no injury at all done to the metal below. The rust which is now so plainly observable on these bared spots has been caused by the action of the salt water reaching the metal after the removal of the varnished coating, as when these plates were examined they were still wet with salt water, having been just taken from Mr. Parsons.

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Mr. Parsons. the river. This exception, of course, does not apply to the specific cases stated by the author, where some of the coatings had actually failed and allowed the water to penetrate and reach the metal. The speaker's observations at the time the plates were removed from the water convinced him that the "Sabin Process" coating was unquestionably the best.

Mr. Stiles. W. M. STILES, Esq.—In a pipe coating applied hot, the material not being thinned down with any vehicle like turpentine, it would be interesting to know whether its stable condition may not be due largely to the fact there is nothing about it to evaporate. There are no holes left in the film, such as are produced by the evaporation of volatile oils in most paints, which is shown quite plainly in some experiments relating to electric insulation. If a little water is placed on the paint so as to afford a large surface of electric contact, the current will pass readily through most coverings that are not baked or do not contain a large amount of resin, which seems to make a continuous, glossy surface.

It would also be interesting to determine whether the baking of the thin coat of oil and resin, asphalt or gilsonite has any greater tendency to unite the materials chemically than the heating in mass in process of manufacture. This is apparently true, judging from the action that takes place in thin layers of such substances when they come in close contact with the air.

Mr. Sherrerd. M. R. SHERRERD, M. Am Soc. C. E.—Experience with the asphalt coating on the steel pipes of the East Jersey Water Company indicates that there are substantially two coatings formed by the process used. The steel is first heated to about 300° Fahr., then dipped in asphalt at about the same temperature, and finally lifted from the bath and allowed to dry. This operation is repeated a second time. During the heating of the steel before it is dipped, an important change takes place on its surface, the rust being changed to a higher oxide by means of a lower oxide produced by the heat. The chemical equation for the change is $Fe_2O_3 + FeO = Fe_3O_4$. The Fe_3O_4 is the stable magnetic oxide. It is of a brownish color and approaches a pasty consistency. In uncovering some of the first Newark pipes protected with the asphalt coating, the latter was found to be intact. If scaled off, the brown paste was exposed, which remained unchanged when uncovered to the air for some weeks, indicating that further rusting of the steel seemed to be prevented.

Mr. Upham. R. D. UPHAM, Assoc. Am. Soc. C. E.—Experiments have been made under the speaker's direction for about a year past on a method of protecting iron ships from rust and from fouling in salt water. These experiments were conducted both on iron plates exposed to salt water and on a sea-going vessel. The material used was a compound of asphalt and copper filings, or copper in a finely divided condition, mixed

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when hot, as asphalt paving mixture is prepared, and applied while still hot with a trowel, as a plasterer spreads plaster on a wall. When cold it forms a sheathing of great toughness and durability, and is sufficiently elastic to adapt itself readily to the expansion and contraction arising from changes in the temperature of the surface to which it may be applied. Mr. Upham.

Two principal objections to the use of copper for sheathing sea-going vessels have been the galvanic action which follows when copper and iron come in contact with salt water, which action destroys the iron, and the liability of water penetrating beneath the sheathing, thereby rusting the iron.

In these experiments the plate or vessel was first painted with a priming coat of pure asphalt to insure the perfect adhesion of the sheathing material. The asphalt in the compound then applied insulates each particle of copper from each other particle and from the iron. The coating was about $\frac{3}{32}$ in. in thickness, and presented a metallic surface, smooth and impervious to water. There has been no evidence of galvanic action, the surface of the sheathing composition was perfectly clean after four months, which was the longest test, and the iron showed no evidence of rust. In addition, the composition was in no wise affected by the water.

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CORRESPONDENCE.

Mr. Crowell. FOSTER CROWELL, M. Am. Soc. C. E.—The experiments described in the paper cannot fail to awaken keen interest among those who have the care of structures exposed to sea water (including, of course, ships), and incidentally in connection with the general subject of painting metallic structures. The experiments, although not conclusive, pretty well establish in general the worthlessness of ordinary painting, as commonly practiced, for the protection of iron thus exposed, and the comparative values of the few descriptions of coatings that proved effectual, in a greater or less degree, in withstanding the effects of sea water.

The only tested preparations that appeared to come within the latter category were the "Durable Metal Coating," spar varnish, and the "Sabin Process" japan. All the others proved practically useless, and this was especially true of the coatings into which lead entered, the most worthless of all being that which is almost universally used at the present day for the painting of iron hulls, viz., a coating of red lead and oil. White lead was turned to a dense black from the sulphides in sewage present in the sea water, and very probably this same cause will account for the rapid destruction of the red-lead coating, which otherwise, as in pure sea water, might not have succumbed so readily or completely. The fact remains, however, that the experiments condemn the use of red lead in comparison with any other of the preparations tried.

Of the three coatings above mentioned as effectual, which word must, for the present, be taken in a comparative and modified sense, the "Sabin Process" specimens in all the classes showed much the best; so far as these experiments go, in fact, the plates of commercially pure aluminum appeared to have been perfectly protected by the japanning, the appearance of the plates containing alloys being less perfect. Generally speaking, the purer the metal, the more perfect the protection, which may be attributable to the fact that the alloys are means of producing electrolytic action.

The spar varnish specimens showed, at the time, the next best results, and these, too, were in a measure proportional to the purities of the plates. Recently the writer examined again a number of the same plates and verified the above conclusions, which had been reached by him at the time of first taking the plates out of the water. He found, on the whole, that the "Durable Coating" showed very much better, relatively, than it had, and seemed to divide honors with the spar varnish. As the latter had at first appeared to cling more tenaciously to the surface of the metal, the suggestion is made that a combination

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of separate coats of spar varnish and "Durable Metal Coating" might prove more efficacious than either alone. Mr. Crowell.

Perfect continuity, inertness and durability are the essentials, and the writer is of the opinion, which would probably be shared by any one examining the varnished plates, that if the varnish coat be applied directly to the metal before corrosion occurs, and, after hardening, be covered with the "Durable Metal Coating," the best result attainable under ordinary conditions with the substances experimented on could probably be had. But this is not intended to convey the idea of superiority to the Japan coating baked on. It is only a statement of the writer's judgment, based on these experiments, as to the best results to be expected where the surface is such that baking cannot be resorted to.

Having had occasion, about a year ago, to study the subject of pile protection against the teredo in connection with a series of wharves built under his direction in the harbor of Charleston, exposed to sea water which is often in violent motion, the writer requested the author to furnish two examples of coatings for experiment. The coatings were not to be applied to the piles directly, but to sheet-iron pipes encircling the piles. Two test pipes were formed of common American sheet iron with riveted seams, the latter being not intended to be water-tight; one of the pipes was treated with the "Sabin Process," and the other with the "Durable Metal Coating." In each pipe a short section of an ordinary yellow-pine pile was placed concentrically, with an annular space of about 2 ins. all around it, which was filled with sand. The pile section was cut a trifle shorter than the pipe, so that two sheet-iron discs nailed to the ends of the pile section confined the sand tightly. The object of using the sand in the experiment was simply to reproduce the exact conditions under which it was proposed to apply the pipe covering, provided the desired coating could be found. The discs were purposely left uncoated in order to render the test as severe as possible, it being assumed that their corrosion in contact with the edges of the pipe might superinduce the speedy corrosion of the latter and that the effect would extend throughout the pipe. The specimens were then immersed in sea water in Charleston Harbor for a period of three months, being pulled up and examined from time to time, but, apart from that, kept constantly submerged. The first two months, January and February, were comparatively cold. It was reported to the writer that no signs of corrosion or deterioration of the coatings were visible up to March 1st. March was warm. Early in April he had an opportunity to examine them personally for the first time. Both specimens were found to be completely disrupted in that the coatings were loosened from the iron, and the iron was exposed to the full action of the water, while both coatings were useless for further experiment, the baked one appearing to

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Mr. Crowell. be in a worse condition, if anything, than the other, for it could be detached in thin laminæ, like mica, in successive layers over the whole pipe; whereas, the other specimen, while wholly gone in places, leaving the metal bare, yet continued to cling in others, though soft and apparently liable to be washed off.

The experiment just described leads to two conclusions: first, that the temperature of the sea water is an important factor in corrosion; and second, that the character of the metal surface has much to do with the efficiency of the coating, the latter being more of a deduction than a conclusion, except when taken in connection with the experiments described in the paper.

The writer is pleased to learn from the author that the investigations are to be continued, and in warmer waters, where, too, pure sea water can be found, the water in the East River at the Navy Yard furnishing only special conditions of reaction. It may be that the effect of normal sea water may not prove as severe as the latter, but it is the writer's opinion that it will be more so.

Mr. Hobson. R. P. HOBSON, Esq.*—The valuable experiments described in the paper have a two-fold interest, as bearing upon the problem of preventing the corrosion of steel and the problem of preventing the corrosion of aluminum in sea water. The two problems, at the present stage, are essentially distinct. The one relating to steel has an industrial importance commensurate with the cost of maintenance of the bulk of the world's shipping, commensurate almost with the vast value of the shipping itself, affecting intimately the length of life of a vessel. But it is an older problem, and has been the subject of long and tireless research and experiment. The diversity of results, however, shows the problem to be still far from solution, a fact, indeed, universally recognized, and leaving open an important field for further research. The experiments in question introduce into the ranks of preventives the varnishes and enamels described and the process of baking. Their behavior in general appears to be excellent, particularly in regard to adhesion and impermeability, most marked in the test plates subjected to the baking process.

Resistance to the attachment of marine growths appears not to have formed part of the subject of investigation. It would be interesting in the additional experiments referred to if this feature were embraced, conditions being such that the plates would be covered with the usual marine growths. It would further be of interest to know the extent of the practicability of application of the promising baking process, whether it is necessarily restricted to small work, the process being applied after assembling. It would also be interesting to know the limitations imposed by cost for this process and for the varnishes in question, and also in what manner the baking process

* Naval Constructor, U. S. Navy.

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secures its superior results, whether impermeability is reduced, while adhesion is increased. Mr. Hobson.

It is not in the bearing upon the problem of preventing the corrosion of steel, however, that the experiments have their greatest interest. Steel is already in general use for hull construction, and will continue in general use. The question of painting is only incidental, however important the incident; it is governed, and will continue to be governed, by the factors, maximum efficiency and minimum cost. The question of comparative results of different compositions is important, but it is not paramount.

With aluminum, however, the case is different. Its very use for marine purposes hangs on the practicability of preventing corrosion. It is not, at the present moment, so much a question of comparative processes, as one of the availability of any process.

When it is borne in mind that the natural field for aluminum is marine construction, where lightness is a capital requirement, particularly in naval construction, where economy of weight has a vital relation to military qualities; when it is remembered that in small construction, in the hulls of small craft, notably the types of torpedo boats and in hull fittings of all classes, the question of strength is secondary in determining the scantlings, the requirements of stiffness and the safeguard against penetration determining the limit long before the limit imposed by strength is reached, where, indeed, aluminum construction would realize the enormous saving of more than 40% over steel, with aluminum, too, of ready commercial production and reasonable cost; when these facts are borne in mind, and it is then considered that aluminum is barred from this wide and fertile field by the sole fact of its corrosion by salt water and salt air, some appreciation can be had of the importance of research and experiment in the direction of finding preventives.

Moreover, the serious condition which the first aluminum torpedo boat, built by Yarrow for the French government, was found to be in after a winter's neglect, the abandonment in consequence of the purpose of the French government to build of aluminum the five consorts of the first, all to be carried on the deck of the *Foudre*, and other torpedo boats of a similar type, these and other adverse incidents have given aluminum a serious setback abroad, while in the United States, similar unfavorable results in fittings, such as stanchion sockets, air port shutters, etc., on the torpedo boats built for the *Maine*, have confirmed the conclusion arrived at abroad, and aluminum is under a pronounced cloud of disfavor.

A careful examination of the facts will show that the general conclusion and impression are not warranted, though the action taken in the cases in question were probably expedient under the circumstances. The conclusion is in no sense final. Indeed, a comparison

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Mr. Hobson. of the results obtained by the very limited scientific research for the prevention of corrosion of aluminum, with the results of the long, laborious and zealously pursued researches for the prevention of the corrosion of iron and steel in sea water, is wholly favorable to aluminum. The conflicting reports from different sources show the present stage to be still essentially experimental, and to pass unfavorably upon aluminum now would be to condemn without trial.

At the present moment a practical preventive of inadmissible corrosion has not been found, and, indeed, the adoption of aluminum would be unwise; but, as surely as day follows night, aluminum is to conquer a wide realm in marine construction. The experiments described are, then, of special value and interest at the present time of reaction.

Through the courtesy of the author, the writer had an opportunity of examining the test plates. He has also had occasion to examine the corrosion of aluminum in the yacht *Defender*.*

In connection with the present and future experiments, the following observations and suggestions present themselves:

Alloys.—The prominent result of the experiments is that which shows that the corrosion increases as the amount of alloy increases, and that copper, as the alloy, particularly accentuates the corrosion. This result is striking in the comparison of the excellent results of Series III, where the alloy was simply 2% of the commercial impurities, with the unfavorable results of Series II, where the same amount of alloy, 2%, was copper. The interest in this feature is special, for it requires an alloy to develop the best conditions for strength, while among the alloys suited to this purpose, copper is prominent, but is by no means exclusive. Moreover, it was a 5% copper alloy that was chosen on account of its high strength for the Yarrow torpedo boat, and it was from this most unfavorable alloy, exposed to most unfavorable conditions in Cherbourg Harbor, that the adverse conclusions referred to above were drawn.

It is evident that with two metals so widely removed in the electrochemical scale, the alloy of copper and aluminum contains within itself strong elements of galvanic action. Special interest would attach to the determining of the state of the copper in the alloy, and the intimate action that takes place, while it is evident that the alloy for marine purposes should be sought in metals not so far down the electrochemical scale.

Coatings.—The experiments, it is noticed, are restricted to varnish and enamel preparations. The favorable results indicate a field for these preparations for the preservation of fittings, if not for hulls.

* See report made to the Navy Department on her method of construction. This report, together with notes on the use of aluminum in naval construction, has been published by the United States Naval Institute.

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Interest would attach to comparative tests of these preparations and those used on ships' bottoms, also with special preparations thus far made and that may be prepared especially for aluminum. A feature that suggests itself from these experiments is that the insulating qualities, as well as the porosity or permeability, may enter to affect results, and there would be interest in determining them in experiments to be made for comparative results. The generally unfavorable results with the baking process would indicate that the process increases the permeability, while the increase of adhesion counts for but little when galvanic action starts beneath. Interest also would attach to determining whether the paints themselves, or any of their components, enter into the phenomena. Mr. Hobson.

Conditions.—The close proximity of plates of different alloys which varied widely in their galvanic action, affected, in all probability, the results in the individual cases. As far as practicable, while still insuring the same conditions, plates of different alloys should be separated far enough not to affect each other. Even those of the same alloy, having the different coatings, should be arranged so as to affect each other as little as possible.

An analysis should be made as near as can be of the mean sample water of immersion, to determine the special agents, if any, that are present.

If practicable, sets of similar plates and coatings should be tested in different waters, and analyses of the water made in each case.

Both coated and uncoated plates, of the same and different alloys, should be immersed in vessels with sea water of sample kinds, and analyses, qualitative and quantitative, made as follows:

Of the materials of the plates before and after immersion; of the water before and after immersion; of the protecting coatings, where used, before and after immersion. These tests should be made progressively, analyses being made at regular intervals.

In addition, the tests should extend to determining the insulating properties of the coatings, and their permeability or porosity, in different thicknesses.

The object of the experiments outlined is apparent. They contemplate, while determining comparative qualities of different alloys and different preparations, to determine the phenomena that take place, and to find each element that enters and the role and relative degree of importance of each.

The writer has endeavored to have tests made by the chemist at the Navy Yard, New York, of corrosion taken from the *Defender*, and also of a sample furnished by the author of a corroded plate, together with the water in which the corrosion took place in a closed vessel, with a view of determining as far as the limited conditions permitted, what were the insoluble parts and what were the soluble products, and the

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Mr. Hobson. parts they played in the phenomena. A sequence of unusual pressure of urgent current work has prevented the completion of these analyses, and the results thus far are unconclusive.

There is no doubt, however, that the scientific method of solving the important problem of preventing the corrosion of aluminum by sea water is to determine first the exact and intimate nature of the phenomena that take place, and then to proceed, with the knowledge thus gained as a basis, to find the means of prevention. The method is laborious, but the end sought offers large rewards.

Mr. Hastings. L. M. HASTINGS, Esq.*—As a part of the extensive addition to the Cambridge Water-Works begun in 1894, about 23 317 ft. of pipe, 40 ins. in internal diameter, was laid. A part of this pipe, 8 392 ft., formed the force main from the pumps to the reservoir, and 14 924.75 ft. was laid as a supply main connecting the reservoir with the center of the distribution system. This pipe was made of soft steel $\frac{5}{16}$ in. thick, in sheets about 7 ft. wide, and of a length equal to the circumference of the pipe. These sheets were riveted up in the usual manner, four sheets together, making a section about 28 ft. long. Each section was then coated with a preparation referred to in the paper as the Sabin coating. The pipe was manufactured and coated in the city, and was then transported on wagons to the trench for laying. The coating bore the abuse incident to teaming well, the injured portions being painted with two coats of what is now called "Durable Metal Coating."

In order to determine practically what effect water and the soil had on the material of the pipe, and also the value of the coating as a protective agent, a number of cast-iron, steel and wrought-iron plates were prepared for testing.

The plates were from 6 to 10 ins. square; the cast one about 1 in. thick, the plate iron and steel $\frac{5}{16}$ in. thick. Twelve of the plates were uncoated, twelve were given one coat of the Sabin coating, and four steel plates were given two coats of the same material. The reason why two coats were used on some of the plates was that in dipping and baking the plates, owing to their small size, the coating material seemed to drain off, leaving a very thin coating on the plate, and the two coats represented much more nearly the average coating as it was obtained on the pipe.

The plates were first carefully weighed by the sealer of weights and measures on his standard scales. Fourteen of the plates were then placed in the water at a gate chamber in the draught conduit leading to the pumping engines, so that when the pumps were in action, a constant stream of water was passing by the plates, thus subjecting them to the same influence as the interior of a pipe. The water taken through the conduit is a mixture of brook and pond water, of a fairly

* City Engineer, Cambridge, Mass.

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PLATE X.
TRANS. AM. SOC. CIV. ENGRS.
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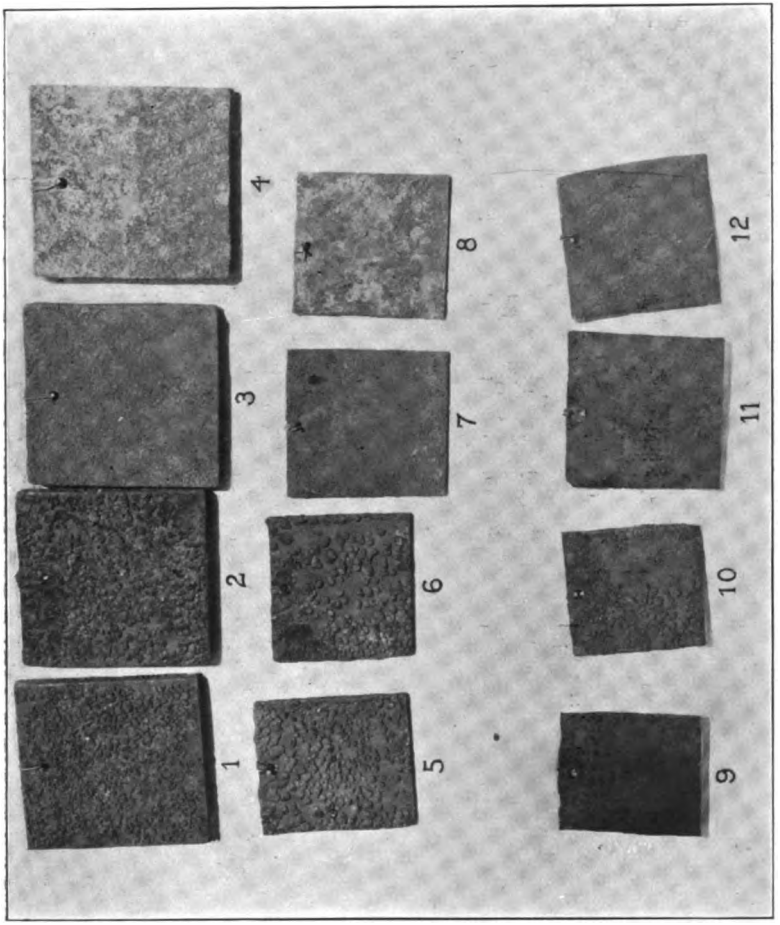
SABIN ON PROTECTION OF STEEL AND ALUMINUM.



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TRIAL TESTS OF UNPROTECTED AND PROTECTED METAL UNDER VARIOUS CONDITIONS, AT CAMBRIDGE, MASS. Mr. Hastings.

Kind of metal.	Coating.	Situation of pipe.*	Original weight.			Weight when examined.			Percentage of increase.	REMARKS.
			lbs.	oz.	dr.	lbs.	oz.	dr.		
Iron.....	Uncoated.....	Water (5).	4	15	1	5	1	4	1.5	Badly tuberculated and rusted; worst of lot.
"	"	" (6).	4	6	12	4	9	0	3.1	Badly tuberculated and rusty.
"	"	Sand (7).	4	15	14	5	1	8	2.0	Very rusty; sand adhered to plate.
"	"	Clay (8).	4	5	10	4	6	4	0.89	Rusty.
Steel.....	"	Water (9).	4	7	11	4	8	8	1.1	Rusty and tuberculated, but not as badly as iron.
"	"	" (10).	4	5	0	4	6	8	2.1	Rusty and tuberculated.
"	"	Sand.....	4	5	6	4	5	12	0.54	Much rust.
"	"	Clay.....	4	9	3	4	9	8	0.4	Somewhat rusty.
Cast iron.	"	Water (1).	20	2	8	20	6	0	1	Badly rusted and tuberculated.
" ..	" . ..	" (2).	19	6	8	19	9	8	0.96	Badly rusted and tuberculated.
" ..	"	Sand (3).	19	10	13	20	0	4	1.7	Badly rusted; sand adhered to plate.
" ..	"	Clay (4).	19	13	10	19	13	8	.....	Somewhat rusty.
Iron.....	Single coat...	Water.....	4	14	11	.....	.....	.....	.....	In good condition; no rust.
"	"	" (12).	4	9	6	.....	.....	.....	.....	Good condition.
"	"	Sand.....	4	9	4	.....	.....	.....	.....	Fairly good condition; some rust spots.
"	"	Clay.	4	13	11	.....	.....	.....	.....	Good condition; little rust.
Steel.....	"	Water....	5	11	11	.....	.....	.....	.....	Fairly good; some little rust.
"	"	"	5	13	2	.....	.....	.....	.....	Fairly good.
"	"	Sand.....	4	11	0	.....	.....	.....	.....	Fairly good.
"	"	Clay.....	5	5	8	.....	.....	.....	.....	Fairly good.
Cast iron.	"	Water....	19	2	15	.....	.....	.....	.....	Fairly good; little rust.
" ..	"	Sand.....	19	4	6	.....	.....	.....	.....	Good condition.
" ..	"	Clay.....	20	8	2	.....	.....	.....	.....	Good.
Steel.....	Double coat..	Water (11)	5	11	13	.....	.....	.....	.....	In good condition.
"	"	" ...	5	6	8	.....	.....	.....	.....	Good condition.
"	"	Sand.....	7	2	12	.....	.....	.....	.....	Very good condition; bright and clean.
"	"	Clay.....	5	8	10	.....	.....	.....	.....	Very good.

* The figures in parentheses refer to the numbers of the plates shown in Plate X.

soft quality, and containing comparatively little acid. Seven of the plates were buried in a sandy soil, and seven plates were placed in a clay soil, all about 4 ft. below the surface of the ground.

The plates were placed in position September 14th, 1895, and were taken out for examination October 8th, 1896. Those which were covered with two coats of the material showed practically no change in appearance, being free from rust, and retaining in some degree their luster or glossy appearance. The plates which had one coat did not look quite as well as those with two coats; slight indications of rust appeared on some. The plates which were not coated at all

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Mr. Hastings. presented a very different appearance, all being heavily coated with rust. Those placed in the water had numerous bunches of rust or solid tubercles on both sides. The plates buried in soil had more or less sand or clay adhering to the plates with the rust.

These plates were carefully reweighed by the same man on the same scales, and showed quite an appreciable gain in weight. The tabular statement on page 505 shows more in detail the facts concerning each plate, as far as the experiment has been carried.

While it is impossible to describe in detail the appearance of every plate, the results of this test may be fairly summed up as follows:

Metal covered with a coating of sufficient thickness was well protected from action of water. The thinner coating did not prove so efficient a protection, while unprotected metal, subject to the same influences, suffered considerably from oxidation and action of the water, the plate iron being affected the most, steel plate to a less degree, and the cast iron the least of all.

Mr. Sabin. A. H. SABIN, Assoc. M. Am. Soc. C. E.—Various questions have been addressed to the author which seem to require an explanation of certain points connected with these tests. One is, why were these three resins, Manilla, Kauri and Zanzibar, selected from the many sorts in use? All these resins are of vegetable origin, being exudations from trees; in no case, however, are these coniferous, so far as the author can learn. There are a few kinds of varnish resins which are used in the condition in which they are taken from the living tree; these are called recent resins, in distinction from fossil resins. The most largely used of these is Manilla, hence this was taken as a representative of this class. Next there is a class of fossil resins of comparatively recent origin, which are dug out of the earth, but were produced by trees of species still living. The resin which exudes from these trees is useless while fresh, but after it has lain in the earth a sufficient time, perhaps for centuries (nobody knows how long), it becomes hard and valuable. Of this sort the most common and typical is Kauri. Finally there are fossil resins so old that the trees themselves have become extinct, and of these the hardest and best known is Zanzibar.

There seems to be somewhat prevalent an impression that these investigations were undertaken with the idea of discovering a paint for ships' bottoms. This is a misapprehension, for the primary object is entirely different. There is much use, in the first place, for a coating for steel and other metals which will protect them from sea water and the moist air near the sea, such as bridges and piers, and metal pipes and other fittings, both on ships and shore; and in a much broader and more general way the author would point out that by immersion in sea water, especially when highly charged with sewage (as in this case) there is an opportunity of subjecting these coatings to extremely

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severe conditions, which allow differentiation between them after a comparatively short test as thoroughly as after a test of several years in ordinary use, with the certainty that all are subjected to the same conditions. This is well illustrated by the fact that the plate with the red lead coating showed so great deterioration of the latter that it had practically lost its value. Now, this was pure red lead and pure oil, and three coats were applied in a more thorough and careful manner than is possible in ordinary work, and to a cleaned and bright surface. This plate would have been in good condition after a very long time if it had been exposed to pure country air, but the coating was practically destroyed during this test. This leads the author to remark on Mr. Crowell's statement that all but two of the coatings tried proved practically useless, which is by no means correct if it is considered that while they may be so for such an extremely severe exposure, though the author is not prepared to admit quite as much as that, some of them showed so much permanence as to encourage their use, not under water, but where they would be exposed to the moist and salt sea air, with, perhaps, occasional wetting, conditions which ordinary paints certainly do not stand well. He is sure that when the results obtained in this way are analyzed they are valuable. For example, the ribs, deck beams and fittings of yachts might be, even if of aluminum, protected in this way. Mr. Sabin.

In regard to the tests on sheet iron described by Mr. Crowell, it may be stated that one of the conditions, made necessary on account of cost, was that ordinary sheet iron should be used. As is well known, this is covered with several thicknesses of black scale, which is intentionally rolled in by the makers, and as it is much thicker on one side than on the other, it is impossible to remove it by pickling. As soon as the sea water obtained access to this scale it rapidly dissolved it, the chemical action going on between the oxide and the saline constituents of the sea water causing a mechanical disruption of the coating, which was no longer of any value. The baked coating came off in flakes, which remained elastic and continuous. The author does not know how to protect ordinary sheet iron under such circumstances, but he thinks if steel plates or beams, which have comparatively little scale, and that removable, were properly treated, the result would be satisfactory. He is now able to produce intentionally such effects as were shown by the baked coating, and to peel off films which show under the microscope a continuous backing of iron oxide, which may then be dissolved off by acid, leaving the film clean on both sides. This illustrates the importance of having a clean metal surface to paint on, a matter which cannot be too persistently reiterated.

The brown coating referred to by Mr. Sherrerd could hardly have been magnetic oxide, because, in the first place, magnetic oxide is black, and secondly, ordinary ferric oxide is not changed by heating

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Mr. Sabin. at 300° Fahr. The author is inclined to think that it might have been a film of asphalt left on the surface, which always shows a brown color, but this is only a suggestion, as he did not see it.

The questions of Mr. Stiles as to whether the baking of the coating effects chemical union among the constituents is of much interest. The speaker has no doubt that it does. For example, the "pipe coating" material, before baking, is readily soluble in turpentine; after thorough baking, it is not softened by prolonged digestion in hot turpentine. It is known that chemical union occurs during manufacture, and its completion during baking doubtless aids to make it permanent. This, in part, answers one of Mr. Hobson's questions. There is no doubt that baking does increase both adhesion and impermeability, provided the coating is composed of materials which are capable of combination, and that the product is an elastic and not a brittle substance. This latter may be said to be in fact the result aimed at in all these experiments; to get a film which is insoluble and unattacked by any chemical agency liable to be brought against it, and tough and elastic, to resist mechanical injury or handling.

The increase of adhesion is probably due in part to the more complete removal of the air film between the surface of the metal and the coating, and this is one reason why a slow-drying coating is better than a quick one, a fact generally conceded. Mr. Hobson's conclusion that baking lessens permeability, the author does not agree with. While he admits that many of the baked coatings were unsatisfactory, it only shows, from his point of view, that these coatings were made of materials which would not stand the greatly increased chemical action produced by baking, and emphasizes the fact, previously realized by him, that making a suitable composition to be applied by baking, so as to get mechanical toughness and elasticity, combined with chemical inertness and permanence in the finished product, is the most difficult thing yet attempted in this kind of work. It seems to him that the pipe-coating material, when carefully applied to a good surface, is satisfactory; but none of the light-colored enamels are so.

He would further point out that no coating can be made which will resist severe abrasion. This should be borne in mind, especially with reference to aluminum. Suppose a coating could be made which would adhere as well as the zinc does in first-class galvanizing, and suppose the coating material as perfect as a film of gold or platinum; this would be generally agreed to be a perfect coating. But if a boat coated thus were to get on a rock, some of the coating would be scraped off. So it will be with anything that can be used, and it is necessary to bear these things in mind when considering the subject. It is one thing to paint steel, another to paint iron oxide, another to paint dirt; but besides all that, it must be remembered that the process is not making the metal incapable of corrosion, but only covering it with a film 0.002 to 0.005 in. in thickness of an inert substance.

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No. 795.

A RESURVEY OF THE WILLIAMSPORT DIVISION OF THE PHILADELPHIA AND READING RAILROAD.*

By GEORGE D. SNYDER, Assoc. M. Am. Soc. C. E.

ABSTRACT.

In 1890 the author was retained by the Philadelphia and Reading Railroad Company to make a survey of its line between Port Clinton and Newberry Junction, a distance of 125 miles. From Port Clinton to Sunbury the road has heavy grades and sharp curves, while from Sunbury to the end of the division the grades are light and the curves easy.

The survey was made to establish a working center line and to introduce transition curves where practicable. All physical features of the ground within a short distance on each side of the center line were located, and data obtained for establishing the right-of-way and property lines. A complete line of levels was to be run over the road, and working grades established for the track, but as the survey was not completed, this work was not finished. From the data obtained by the survey, maps and profiles of the line were prepared on which were

* The paper of which this is an abstract was accepted by the Committee on Publication for filing in the Library of the Society, where it may be examined. Further information concerning its contents may be obtained by application to the Secretary.

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to be recorded all subsequent changes in tracks, alignment and right-of-way. This work, like the leveling, has never been finished.

In running the center line the tangents were established first. If the point of intersection was accessible, the intersection angle and the external secant were measured. With these data the degree of the curve was obtained which would fit between the two tangents and strike the apex of the curve. If an error of one to five minutes was allowable in the degree of curvature the latter was obtained on the slide rule by the formula R = external secant $\div$ tan. $\frac{1}{2} I$ tan. $\frac{1}{2} I$, where R is the radius, and I the intersection angle. This formula is readily solved by the slide rule, which was employed largely in the computations made during the survey.

It was found inadvisable to run the tangents to an intersection where the ground was broken, uncleared or steep, as the measurements on the comparatively level roadbed were more accurate. In such cases the degree of the curve was obtained by holding a tape against the inside of the outer rail at points 62 ft. apart. The length in inches of the ordinate to the rail at the middle of the chord gave the curvature at that point in degrees. The mean of several measurements was assumed to be the degree of the curve. The location of the P. C. was then judged by the eye and a trial curve run, without putting in any stakes, to the end of the curve or until the trial line left the roadbed. In the latter case the distance D, from the trial line to the proposed center line at the last station was measured, and the distance, S, necessary to shift the P. C. was obtained by the formula $S = D \div$ sin. A, A being the total angle to the last station. By means of the slide rule, this shift can be ascertained readily, as well as the amount it will throw the line from the trial curve at each station. In this manner a new P. C. can be obtained and the curve run around to the next tangent.

When the degree of curvature found by means of the 62 ft. chord proved to be in error, the trial curve was generally inside or outside the track at its apex. Sometimes it was found that both the trial P. C. was incorrectly located and the degree of the trial curve was wrong. The data for changing the degree of the curve and shifting the P. C. were furnished by simple computations on the slide rule. Having run the trial curve for eight stations, say, the slide rule was set as explained previously for a shift of the P. C. to make the eighth

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trial station strike the center line; the same setting also gave the distance this shift would move the fourth or middle trial station. The difference between this distance and the distance the fourth trial station had diverged from the center line gave the amount of change in the external secant necessary to make the curve fit the track. The degree of the new curve was then found by the slide rule. To find how far this new degree of curve would diverge from the trial curve and the track, the formula $O = \frac{1}{8} n^2 D$ was used, in which O is the offset from the tangent or from one curve to another, n is the number of 100-ft. stations, and D is the degree of the curve, or the difference in degree of two curves. This formula furnishes the data necessary to complete the location, and is particularly well adapted to the slide rule.

Transition curves were run by means of Froude's formula * $O = L^2 \div 24 R$, in which O is the main offset from the circular curve to the tangent, L is the length of the transition curve, and R is the radius of the circular curve. By substituting $5730 \div D$ for R, the formula becomes $O = L^2 D \div 137520$, where D is the degree of the curve. This curve is practically a cubic parabola having properties which enable all the computations involved in using it to be made on the slide rule. The transition curve bisects the main offset at the P. C., and is itself bisected by the offset line at this point. The offsets to the transition curve from the tangent or the circular curve are as the cubes of the distances, and the deflection angles to points on the transition curve are as the squares of the distances.

In the field, a simple curve which would fit the track was first found, and the offset was either assumed or calculated on the slide rule from an assumed length. If the tangents were offsetted and the same degree of curvature used as before, it would throw the whole curve toward the center a distance about equal to the offset. As this would generally throw the track off the road-bed, the usual practice was to sharpen the curve enough to make its apex strike the same place as before. To offset this the external secant of the new curve was reduced by an amount equal to O tan. $\frac{1}{2} I$, I being the intersection angle of the tangents. This distance was ascertained on the slide rule. From the point of the transition curve to the P. C., the offsets to the transition curve were set off from the tangent. The transit was then

* See Rankine's "Civil Engineering," p. 652.

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placed over the P. C. and the remaining points of the transition curve located by offsets from main curve, the same offsets as before being used, but in an inverse order and an opposite direction.

The intermediate points on the transition curve can also be set by deflection angles. The total angle between the tangents at the beginning and end of the curve is equal to the total angle for the circular arc it replaces, and is obtained by multiplying the degree of the circular curve by half the length of the transition curve. One-third of this product is the deflection angle from the original tangent to the point where the transition and circular curves join. As the intermediate angles are as the squares of the distances, they are obtained readily on the slide rule. The offset method was generally found preferable, except where the transition curve is quite long. The same principles were applied in running one of these curves between two circular arcs.

The advantages of this method are as follows: The engineer is not limited to a certain number of curves with fixed lengths and offsets, as in methods where tables are used, but practically any length of transition curve can be used with any degree of curve. The calculations are all simple, and can be performed readily on the slide rule. The method of staking out and running the curve does not differ materially from ordinary curve work, and any transitman can learn the principles on which the method is based. It does not require complicated methods of plotting and keeping field notes. On original location, it adds nothing to the cost of the field work, as the transition curve need not be run, but merely provided for by making the necessary offsets at each P. C. and P. T., leaving the curve to be run in whenever necessary during construction, which will not be generally until track is to be laid. It will be found in numerous cases to facilitate location, because a long curve need not be rerun merely because it does not quite strike the terminal tangent, as any slight deviation will merely make the offset of the transition curve correspondingly greater or less.

Engineers who have been engaged on rerunning old locations know that the time spent by a party thus employed in actually marking the center line is but a small part of their working hours, and that much of their time is spent waiting for the man in charge to make the necessary calculations. To reduce to a minimum the delay caused

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by these numerous computations, the author began the use of the slide rule. It was first employed in checking results, and, as confidence in it increased and a knowledge of its possibilities and limitations was obtained, some calculations were undertaken on it alone, and finally practically all of them, with the result that the party had little idle time.

The plotting was done very carefully. The maps were drawn on a scale of 1 in. to 50 ft., one-half mile being on each sheet. A longitudinal section of each bridge was shown, on which the general dimensions of the structure and its members were marked. The adjustment of the right of way was very difficult. The land was of slight value when the road was built, and little care was taken in drawing up the deeds. Many of them merely granted a right of way so many rods wide through a certain property in a certain township. As many of the deeds were drawn up in 1830, and there had been numerous changes in the ownership of adjoining properties since then, the title of nearly all the real estate along the line had to be traced back from forty to sixty years. All information relating to the right of way was tabulated on separate sheets, and a reference number on each separate purchase on the map showed on which sheet this information could be found. Index maps on a scale of 1 in. to 2 000 ft. were also prepared, as well as sketch maps of each train dispatcher's district. On the latter were indicated the main track, branches and all sidings, showing the number of cars each would hold.

The paper is accompanied by fifteen drawings, two photographs and an appendix giving slide rule settings applicable to field calculations on a railway survey.

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No. 796.

THE PROPER PROFILE FOR RESISTING WAVE ACTION.*

By ROBERT FLETCHER, Assoc. Am. Soc. C. E.

ABSTRACT.

In the correspondence contributed by the author to the paper† by Theodore Cooper, M. Am. Soc. C. E., entitled "Some General Notes on Ocean Waves and Wave Force," attention was drawn to the theory of waves advanced in 1831 by Col. A. R. Émy. The most novel feature of this theory assumes the existence under certain conditions of a bottom wave caused by the oscillatory waves nearer the surface. This bottom wave is a true wave of translation, which has little or nothing of an oscillatory character, and must exert horizontal energy nearly in proportion to its mass and the square of its horizontal velocity. The present paper is a continuation of the correspondence referred to, and discusses the best profile for resisting wave action.

One of the most common modes of failure of sea walls with vertical front is the undermining of the foundations, or the wearing away and breaking up of the lower courses. Nearly every writer upon the sub-

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†See p. 161.

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ject makes mention of the common failure at the foot of the works. This is often said to be caused by the "undertow," or the suction and downward motion of the water at the wall.

Rankine[14] proposes to moderate the undermining action by forming the face of the wall into steps, "so as to interrupt the vertical descent of the waters." Stevenson's experiments on a stepped wall show that on such a profile the force of the waves may be increased six times. If such construction is sufficiently massive, with joints absolutely impenetrable by water or air, it may offer an efficient check; but with the usual imperfections in such work, it would appear that the wave of translation has simply more points of attack, and that the destruction of the lower courses is likely to be rather hastened than retarded.

Émy advocated a profile which shall present no sudden obstacle to the rapidly moving bottom wave, but shall deflect it from its horizontal movement, and compel it to expend its energy in raising itself and the superincumbent weight of water. While the cycloid was proposed as specially suitable, the form is not of chief importance, provided the curve is tangent to the bottom and of a radius sufficient to promote gradual deflection of the largest bottom waves.

Émy rebuilt the sea defences of St. Jean de Luz, at the head of the Bay of Biscay, by a long incline. A former expensive breakwater there, built of concrete and stone, with an average front slope of one vertical to three horizontal, and with a footing of heavy stones confined by piles just below low-tide level, was completely destroyed, chiefly on account of this provision. These rocks, some of which weighed 4½ tons each, acted as missiles, impelled by the furious bottom waves, and made the destruction far more rapid and complete by a veritable bombardment.

The use of heavy material and steeper slopes is perhaps the most common way of opposing wave action. Rock is dumped into the sea and allowed to find its place. Admirably adapted to some localities, where large stone is plenty and where the wave energy is not great enough to move the stones, this system is generally used; but such works are often very expensive, require years of repairing before any degree of permanency is secured, and may be broken up again by the next great storm. The use of small stones in such cases invites failure. Half a century ago the Plymouth breakwater had changed its form

[14] Rankine's "Civil Engineering," p. 761.

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three times, and each time its base was extended and sectional area increased. At Cherbourg the same thing had happened, and the breakwater was said to have a base 350 ft. wide.

In some reports of the Corps of Engineers, U. S. Army, the question of foundation is dismissed with the statement that anything capable of bearing the weight of the upper part will do up to within 15 ft. of low water, because there is little action below that depth. This reasoning has led to the use of small material for the lower slope and to reserving the heavy blocks for the upper slope. The facts are that on most breakwaters built of *pierres perdues* not reaching far above sea level, the material from the seaward slope is carried over to the harbor side by the waves. For example, at Plymouth stones weighing from 3 to 8 tons each, firmly fixed in the pavement of the seaward slope, were torn up and projected over the breakwater into the sound.

At Delaware, before the completion of the breakwater, a stone weighing 7 tons was moved 18 ft. from the outside to the inside of the ice-breaker, and 200 tons of heavy stones, which had been well wedged together, were swept over to the inner side.

Vertical walls have been built on large mounds of stone, to prevent the passage of material from the outside to the inside, but it is evident that they may be subjected to heavy bombardment, and are never quite safe from settlement if the loose stones of the foundation are displaced. Émy designed walls with a curved face tangent to the bottom and having sufficient radius to deflect the largest waves.

Although these walls had existed but ten years at the date of his book on waves, they had proved much more durable than the old-style walls.

Émy was not the first to propose the curved front. Franz Gerstner, of Bohemia, had recommended it previously, and, later, English engineers have decided in favor of it, and the *Proceedings* of the Institution of Civil Engineers contain many discussions as to the merits of vertical, inclined and curved profiles.

For works designed primarily for other purposes than to hold the sea in check, as sea walls, lighthouses, etc., J. Scott Russell held the concave profile to be unquestionably the best form, for it approximates to the line of least resistance. Works rising above the surface of the sea, and designed to stop the waves so as to produce comparative calm, should exert a uniform action. To accomplish this,

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he says: "The cycloid is the approximate form, and especially so when the materials are weak and not abundant." If there is a profile which will enable weak materials to do the work of strong, the question may well be raised whether it is good practice to use any other form.

Mr. Russell urged objections, of which a chief one is the fact that the water is forced up higher than is the case with a vertical wall, and that the parapet is more likely to be torn off, but this distinguishing feature would seem to be an advantage, for the force which raises the water would have been expended upon the wall itself. The parapet, if built smooth with no projections, will suffer no great impact, and may be curved slightly forward so as to throw the water seaward. Another objection is that the pressure is thrown higher up the wall, but it would seem that this is preferable to lower action on vertical walls, which destroys the foundation and causes the failure of a large part of the work.

Having referred to Mr. Russell as an advocate of the cycloidal profile, it must be said that he considered it useless and injurious for certain kinds of waves which he believed to exist. He thought that the rollers which occur at the Cape of Good Hope and other places, and the common ground swells, have a forward motion *en masse*, the whole depth of water rushing forward and then receding. This agrees with the observations of Sir John Coode at the harbor of Colombo, who stated that water is forced up 120 ft. above sea level by waves which are only 15 ft. high at the moment they strike the rock. He believed that during the occurrence of the heavy monsoon swell, it was the swell itself, not a broken sea, which came in with a great roll. On the other hand, these swells in mid-ocean do not have any perceptible motion of translation, but are taken as the true type of a wave undulation.

That these immense swells really entrain very large masses of water, is likely; if so, the bottom waves may become so large as to constitute the greater part of the whole wave, and, in shallow water, operate to impart to nearly the whole mass of the entire wave an energy horizontally directed. Mr. Russell would use a long reversed curve, concave above and convex below, to resist such action. The abrupt rise at the bottom would destroy much of the momentum of the bottom wave, but it would be subject to a large share of the shock that destroys the footing of vertical walls.

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At Aberdeen the pier head was first constructed as a vertical wall in shoal water, but, "the foundations being insecure," it fell more than once from the "constant percussion of the undertow." "The cycloidal form was then adopted, by which a gradual resistance was offered to the force of the waves, and the head has stood for half a century" (reported in 1860).

Concave walls have been built near Edinburgh which have had varying success, and have been pointed to as throwing doubt on the practicability of that design. Nothing can be proved conclusively by them, because they were built of very light materials, and had other defective features. Still, compared with the massive vertical walls in the neighborhood, costing many times as much, they have done good service. The south pier at Whitehaven, built with a curved face and an apron of large stones founded upon sand, has resisted many great storms. Mr. Scott Russell cited this case as gratifying to him because of its success, although only an approximation to his recommended profile.

Lighthouse builders have made use of the concave profile because of its great stability, and, although not designed primarily to deflect waves of translation, the action at Eddystone and at other lighthouses shows fine examples of the manner in which it deflects and directs the energy of the waves vertically instead of allowing them to act horizontally against the structure.

Among modern French engineers the curved profile is not held in general favor. Some four years ago the author addressed an inquiry to M. Guillemain, Director of the National School of Bridges and Roads of France. The latter quoted the following opinion of M. Laroche, Engineer in Chief:

"Experience appears to justify curved fronts in a certain number of particular cases, and for special works exposed to less violent waves which have been broken by their passage over the incline of long submarine beaches of gentle slope; also, a curved front at the summit of slopes of shifting beaches for defence.

"Per contra, practice seems to have shown that for great jetties exposed to violent seas, the curved form of exterior front does not palliate the shock of the waves, at least during the continuance of real tempests.

"In fact, during a gale, the billows do not approach a jetty under the hypothetical conditions upon which some have reasoned. It is no longer a layer of water which glides upon the slope of the sub-

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structure, as does the billow which exhausts itself upon a beach of gentle slope; it is an enormous mass of water, a veritable liquid hillock, of which the ridge, mounting above the sea, precipitates itself with a crash upon the work, and strikes almost at the same instant the whole height from the parapet to the top of the substructure. Under these conditions it is conceivable that a curvature of the front, a curve always rather slight at best, can have only a small influence upon the violence of the shock of the billow.

"Indeed, jetties with a curved front require, in order to insure the stability of the parapet, a cube of masonry quite as considerable as jetties with a straight front, and the billows surmount as well the former as the latter, while the tempest lasts. So that curved fronts appear to have advantages only in tractable weather, that is to say, when one can conveniently exempt one's self from having recourse to a disposition which leads always to costly requirements in execution.

"The theory of curved fronts requires quite a sharp angle at the base of the superstructure, so that the wave may reach it without shock, and this condition had been at first observed at St. Jean de Luz; but this thin part, lacking in solidity, was repeatedly broken and destroyed. It has become necessary to commence the curve immediately above the blocks of the substructure, and to give at the base of the superstructure a vertical front of 2 m. height.

"Finally, and within the limits which permit us to formulate an opinion of by no means general application upon questions so complex, so controverted, and where, moreover, local and special circumstances play so important a part, we can state that the form of the exterior front of the superstructure of a jetty is indifferent from the point of view of the resistance of the work during tempests. It is expedient, then, in most cases to adopt the more simple and more easily realized form, namely the plane front."

M. Guillemain adds: "The opinion of M. Laroche is that of the greater number of French engineers, and all breakwaters which have been recently built in France and in Algeria, with rare exceptions, have received plane fronts rather than curved."

It appears certain, however, that the curved profile has, by actual comparative trials, proved more efficient and permanent than the plane front in some situations; and that it should be used under those conditions which are favorable to its application. For example, on rock bottom at or not far below low-water level, the tangential junction of the curved toe with the rock may be properly made, and if the lowest courses are dowelled and clamped together and the joints of the entire front made water-tight as low down as possible, a much smaller cross-section would be required because of the relief from direct impact

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afforded by the deflecting curve where the waves are not too severe. Other favorable conditions and dispositions will be recognized by the experienced engineer.

It is hardly conceivable that there is any case where the body of the sea comes upon a work *en masse*, with a common horizontal velocity throughout, excepting that of the earthquake wave, which, fortunately, is not common, and which man should not expect to oppose successfully by any work which he has the means to build. This is a great sway, backward and forward, like that produced in a tub of water which is tilted. The tidal bore is not to be classed as a wave to be resisted by a breakwater.

The history of jetties and breakwaters is largely one of failures, costly and often repeated. Too much has been done in many places without a sufficient study of past experience, or an adequate idea of the enormous energy and the mode of action of storm waves. Witness on a very small scale the utterly inadequate means and methods of shore protection adopted at some seaside resorts, as Long Branch and Coney Island, where, if it is true that sufficient works would be too costly, it is no less true that there should be enough appreciation of the task to deter from such futile endeavors.

Nevertheless the judgment of the engineer must be largely guided by a knowledge of such failures, and it is to be regretted that the record of failures and damages to such works is not more complete, especially as relating to jetties and breakwaters on American coasts and lake ports.

The standard type of breakwater adopted by the Corps of Engineers, U. S. A., for the harbors of the Great Lakes is a timber crib of rectangular cross-section, filled with stone and resting usually on a broad base of random stone which the waves have compacted into a more or less stable profile. While generally doing good service for that type of construction, which is approved as usually the best within the means available for construction and maintenance, these works have not proved altogether satisfactory, having been repeatedly destroyed at certain points in various harbors. While the writer would not presume to make direct criticism of the system so long adhered to by distinguished engineers, he considers it a fair question whether the principle of the curved profile is not especially applicable in such exposures as are found on the Great Lakes. Cribwork has been and may

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readily be built with a curved front and the face protected by planking below low water and as far above as desired. Such a mass is peculiarly subject to the disintegrating action of water pressure transmitted through all the interstices, and it would seem that the double-planked concave front would give such deflection to the storm wave as would greatly mitigate the shock.

Mr. W. R. Kinipple calls attention* to the great value of monolithic work of concrete for breakwaters. His experience of thirty years has convinced him that such construction, carried down even to the bed of foundation, and including all the substructure by a process of injection and "stock-ramming" of cement grout, embodies the true principle, by avoiding all the difficulties of joints, dowelling, clamping, dovetailing, and the inherent weakness of a substructure of separate fragments, where the conditions are especially suitable for destructive action of violent waves. He has successfully reconstructed and extended important works by this system, and claims greater efficiency, durability and less cost than for any form of jointed structure. It is needless to add that concrete is particularly well adapted to the execution of a curved profile.

* See *Engineering*, Vols. I and III.

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MEMOIRS OF DECEASED MEMBERS.

ALEXANDER DALLAS BACHE, Hon. M. Am. Soc. C. E.*

DIED FEBRUARY 17TH, 1867.

Alexander Dallas Bache, born at Philadelphia, July 19th, 1806, was the grandson of Richard Bache, Postmaster-General of the United States from 1776 to 1782, and a great-grandson of Benjamin Franklin. He was educated in a Philadelphia school and at the United States Military Academy at West Point. He was graduated at the head of his class, one of unusual ability, from the latter institution in 1825, and remained there for one year as an assistant professor. Subsequently he was assigned to duty at Newport, R. I., under Colonel Joseph G. Totten, Hon. M. Am. Soc. C. E., and while there married Miss Nancy Clarke Fowler.

Leaving the army he went to Philadelphia as Professor of Natural Philosophy and Chemistry in the University of Pennsylvania, and then began a career as physicist and astronomer which soon brought him an international reputation. At that time the Franklin Institute had not been established very long, and he took an active part in the work which raised it to a prominent standing among scientific associations. He was also interested in the American Philosophical Society. In addition to the work connected with these societies and with the university, he was engaged in conducting investigations at his private observatory, where he determined for the first time in this country the periods of the daily variation of the magnetic needle, and by another series of observations the connection of the fitful variations of the direction of the magnetic force with the appearance of the aurora borealis. In 1836 he was elected President of the Board of Trustees of Girard College, and went abroad for the school to investigate European educational methods. This position he resigned six years later, as well as his connection with the free public school system of Philadelphia, and the High School of that city, which he had organized.

Although Prof. Bache's investigations in subjects connected with terrestrial magnetism had been of value to engineers, he was brought into more intimate relations with them in 1843, when he succeeded Mr. F. R. Hassler as Superintendent of the United States Coast Survey. What he accomplished in this office is indicated in a general way by the following extracts from a resolution passed on February 18th, 1867, at a meeting of his former assistants :

"Whatever of excellence there may be in the extended system of operations now carried on by the coast survey on every portion of our

* Memoir prepared from information furnished by A. T. Mosman, M. Am. Soc. C. E., and from papers on file at the House of the Society.

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coast is due to Prof. Bache. He came to the charge of the work at a time when its operations were conducted upon a small scale and restricted to a limited portion of the coast. In a wonderfully short space of time he succeeded in winning the confidence of his official superior, and in securing the consent of Congress to a gradual enlargment of the work to its present scale. He called to his assistance men of thought and men of action from civil life, and from the army and navy, and with a rare felicity, discerning and applying the special aptitudes of each individual, he wrought out from discordant material a harmonious whole.

"He combined high administrative ability with vigor and energy in execution. While allowing and inviting free criticism of his plans during their inception, he exacted a rigorous accountability from the officers intrusted with their execution. Discipline under his administration was none the less real that it was not apparent.

"Prof. Bache was eminently just. The Coast Survey reports—those monuments of his fame—are full of evidence of the scrupulous care with which every officer serving under him received proper credit for his labors. His quick and ready appreciation of merit in every department of scientific inquiry and action, whether theoretic or practical, has been felt through the entire country, and has been of lasting benefit. To his fostering care and aid we owe the present perfection of the telegraphic method of obtaining longitude which recently achieved its crowning triumph in the determination by the Coast Survey of the precise difference of longitude between any two points in Europe and America, through the Atlantic cable."

Prof. Joseph Henry said, in his eulogy of him:

"When Prof. Bache took charge of the Survey it was still almost in its incipient stage, subjected to misapprehension, assailed by unjust prejudice, and liable, during any session of Congress, to be suspended or abolished. When he died, it had conquered prejudice, silenced opposition, and become established on a firm foundation as one of the permanent bureaus of the executive government."

His unusual administrative ability has been referred to particularly by most of his biographers. Prof. Fairman Rogers wrote* that he loved to put the machinery of his office together, wind it up, and then dismissing it all from his mind, hear the report at the designated time, when he would take up the thread of the matter just where he had left it last, as if he had thought of nothing else during the interval. "He understood thoroughly the way of doing nothing for himself that could be done for him by others, and thus reserved his time and powers for that work which he alone could do." Nevertheless, his early training in the details of observations was so thorough that interpolated figures are reported to have rarely escaped his notice, and his ability to tell an observer that on such a night he had omitted to examine the level of his instrument was little short of marvelous.

The Coast Survey was only one department of government work with which he was connected, for he was also, at one time or another, Superintendent of Weights and Measures, Lighthouse Commissioner and member of the Lighthouse Board, Regent of the Smithsonian Institute, and a Vice-President of the United States Sanitary Commis-

* *Journal* of the Franklin Institute, May, 1869.

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sion. The American Association for the Advancement of Science was founded largely through his influence, as was the National Academy of Science, and he was a president of both these organizations, as well as the American Philosophical Society. Foreign scientists recognized the importance of his work at an early period of his career, and he received many honors from European associations. His published works are many, and a list of them, outside of his official reports, would be too voluminous to print here.

At the request of the Governor of Pennsylvania, although overwhelmed with other public labors, he planned lines of defenses for Philadelphia during the civil war, and, to a certain extent, personally superintended their construction. Unaccustomed for many years to direct exposure to the sun, this work proved too much for his physical strength, and brought on the first indications of that malady which terminated his life.

Prof. Bache died at Newport, R. I., February 17th, 1867, and was buried in the Congressional Cemetery at Washington.

He was elected an Honorary Member of the American Society of Civil Engineers on March 2d, 1853.

ÉMILE MALÉZIEUX, Hon. M. Am. Soc. C. E.*

DIED MAY 20TH, 1885.

Émile Malézieux was born at St. Quentin, France, June 8th, 1822. He entered the École Polytechnique in 1841, and afterward the École des Ponts et Chaussées, from which he graduated in 1846, with next to the highest honors in the class.

After a short tour in England, he was placed in charge of a division, first of construction and later of maintenance, of the canal from the Marne to the Rhône, in the Departments of the Marne, the Meuse and the Meurthe. He held this post for twelve years, serving also during a part of this time on the construction of a portion of the railway from Paris to Strasburg, within the Department of the Meuse.

In 1859 he was assigned to duty in Paris in connection with the construction of the Belt Line Railway, a position he held for ten years. Toward the end of the next year he was also in charge of the canalization of the Marne, in the Department of the Seine, including the construction of the St. Maurice Canal and the Joinville Dam. At the Paris Exposition of 1867 he received a gold medal for the type of metal lock gates introduced by him on the St. Maurice Canal.

In 1868 Mr. Malézieux was appointed to a professorship at the École des Ponts et Chaussées, and was given the course in interior navigation,

* Memoir prepared from papers on file at the House of the Society.

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which he retained until 1877. His knowledge of public works led to his being sent in 1870 to this country to examine the works of American engineers. Here he collected a large amount of data, which was subsequently employed in the preparation of his two-volume report, entitled "Travaux publics en Amérique, en 1870." This was so satisfactory that the author was sent three years later on a similar mission to England, which resulted in the publication of another valuable report. These labors were not allowed to interfere with his regular work as a teacher, and, indeed, his original course was supplemented in 1871 by another on general methods of construction.

Early in 1877, his connection with the school was closed, and he was unanimously elected Secretary of the General Council of the Engineering Corps (Ponts et Chaussées), where he served with distinction until 1882. Then his health, feeble for several years, broke down, and he was given an indefinite furlough in which to recover. This enforced rest, although fully earned by continuous service since 1846, proved irksome to such an active man, and he applied for reinstatement. This was granted him, and his subsequent work showed no diminution in excellence. His death occurred suddenly on May 20th, 1885, his rank in his corps at the time being that of Inspector-General of the Second Grade.

Mr. Malézieux was elected an Honorary Member of the American Society of Civil Engineers on November 3d, 1880. He was made a chevalier of the Legion of Honor in 1856, and an officer in 1874. He was also a member of the American Philosophical Society of Philadelphia and the Academic Society of St. Quentin.

GENERAL JOSEPH G. TOTTEN, Hon. M. Am. Soc. C. E.*

DIED APRIL 22D, 1864.

General Totten was born in New Haven, Conn., on August 23d, 1788, and graduated at West Point as 2d Lieutenant of Engineers on July 1st, 1805. Resigning in 1806, he served as Secretary to Captain Jared Mansfield, then Surveyor-General of Ohio and of the Northwest Territory. He re-entered the Corps of Engineers as 2d Lieutenant, February 23d, 1808, and then served in that corps until the end of his life.

The war of 1812 furnished full opportunity for the display of his ability. He was assigned as Chief Engineer to the army under General Van Rensselaer, and was engaged in the battle of Queenstown in 1812. He was then transferred to the Army of the North as Chief Engineer under General Dearborn, and was engaged in the capture of Fort George. In 1814 he became Chief Engineer of the army on Lake Champlain under Generals Izard and Macomb, and was engaged at the

* Memoir prepared by Gen. C. B. Comstock, M. Am. Soc. C. E.

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battle of Plattsburg, a place which he had fortified. Previously brevetted Major, he was brevetted Lieutenant-Colonel for gallant conduct at Plattsburg.

From 1817 to 1819 he was the Superintending Engineer of fortifications at Rouse's Point, N. Y., and then became a member of the Board of Engineers charged to prepare plans for defensive works for the sea coast of the United States. Its members were then General Bernard and Lieutenant-Colonel Totten. General Bernard was an able officer of the French Corps du Génie, who was Assistant Engineer in our service from 1816 until 1831, when he returned to France, and subsequently became Minister of War. Although naval officers were sometimes associated with the Board of Engineers, as also the resident engineers of local defences, it was on those two, mainly, that the planning of the defensive works on our seaboard devolved, and a large part of the work was done by General Totten.

The system was prepared and was largely executed in a period when a 42-pounder was one of the heaviest guns on shipboard, and when all naval guns were behind wooden walls. It consisted in defending our harbors by a number of guns which should be more than a match for any probable naval attack, these guns being of power equal to those in the English and other foreign navies, and placed behind masonry walls which were impenetrable to the naval fire of the day, or behind other effective cover. In similar foreign defences and in the earlier ones of this period built by us, the embrasures were large, and the gunners were seriously exposed to small projectiles. General Totten devised in the latter part of the period a shutter embrasure lined with masses of wrought iron which largely reduced the embrasure opening, and correspondingly diminished the exposure of the gunners. It was an improvement of the highest value, and in 1855–60, when the plans of defence had been carried far toward completion, our system had no superior in the world, and the credit therefor was largely due to General Totten. But at that time, with the gradual introduction of rifled guns of large caliber, and with the possible use of armor on shipboard, the naval vessels of the world, with their wooden walls, became things of the past, and sea-coast defences had to change to meet the new navy.

From 1838 until his death General Totten worked unremittingly at the duty laid upon him of adequately defending the coast of the United States. No question was too large to be considered, no detail too small to be examined by him. His letters were filled with drawings from his own hand, and, in their terseness, clearness and precision, were models for others.

From 1825 to 1838 he was Superintending Engineer of the construction of one of the largest defences in the United States, namely, Fort Adams, R. I. In 1838 he became Colonel and Chief Engineer, U. S.

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Army, and thereafter his station was in Washington. With this change his duties changed so as to embrace the whole United States. They were so performed as to gain the admiration of all save unscrupulous superiors who could not bend him to their schemes.

In the war with Mexico, General Scott, who had the highest opinion of his ability, made him his Chief Engineer during the siege of Vera Cruz, and he was brevetted Brigadier-General for gallant and meritorious conduct at the siege.

In 1862 he was member of a commission to report on the defensive works at Washington and Alexandria.

In 1863 he was made Brigadier-General; on April 21st, 1864, was brevetted Major-General, and on April 22d, 1864, his useful and noble life ended.

From 1846 until his death he was a Regent of the Smithsonian Institution; a corporator and member of the National Academy of Sciences from March 3d, 1863; and a Member of the Light-House Board from its organization in 1852.

He devoted to his country's service great ability, unswerving honesty and unremitting labor. Gifted by nature with an even, well-balanced mind, he was a courteous gentleman, and, what is deeper, he was a good, kind-hearted man.

General Totten was elected an Honorary Member of the American Society of Civil Engineers March 2d, 1853.

SQUIRE WHIPPLE, Hon. M. Am. Soc. C. E.*

DIED MARCH 15TH, 1888.

Squire Whipple, the son of James and Electa Johnson Whipple, was born in Hardwick, Mass., on September 16th, 1804. His father was a farmer and later the owner of a small cotton mill, and the son spent the earlier period of his life in work connected with the farm and the mill. For about six months in each year he attended the district schools of Hardwick until he was thirteen years of age, at which period, or in 1817, his father removed to Otsego County, N. Y, and the son again assisted him in farming. After teaching school for a time Squire Whipple attended the academical schools of his neighborhood, and in 1829 he entered the senior class of Union College, at Schenectady, N. Y., and was graduated one year later, with the degree of A. B., from that institution.

His early connection with the machine shops of his father's mill at Hardwick had assisted in developing a natural inclination towards mechanical pursuits and had to a certain extent guided him in his

* Memoir prepared by D. McN. Stauffer, M. Am. Soc. C. E.

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studies. The construction of the Erie Canal and other public works nearby induced him later to enter the new and then very vaguely defined field of civil engineering, and, upon leaving Union College in 1830, he sought occupation of this character. He first found work as a rodman, and later as leveler, upon the Baltimore and Ohio Railroad; but his service with that company seems to have been short, and he was chiefly occupied for some years under Holmes Hutchinson, late of Utica, N. Y., in fixing the boundaries of land necessary for the construction and maintenance of parts of the Erie Canal, and in surveys and estimates for the enlargement of this waterway. In 1836–37, he was for a short time resident engineer on the Eastern Division of the New York and Erie Railway, under the late Edwin F. Johnston, as chief engineer; but work was soon suspended upon this enterprise and he was out of employment. With the exception of five or six surveys and reports upon projected railways and canals, submitted between 1837 and 1850, the list here mentioned constitutes about all of the experience of Mr. Whipple in general field practice.

In the intervals of enforced leisure Mr. Whipple turned his attention to the manufacture of engineering field instruments; and about 25 leveling instruments and several fully equipped transits were made by him at this time. In 1840 he brought out his first invention of any note in the form of a scale for weighing canal boats of from 200 to 300 tons. He made a model and later built the first enlarged weighing lock scale ever constructed upon the Erie Canal. This scale worked satisfactorily and served as a model for others built by Mr. Whipple and those following his designs.

Mr. Whipple's first patent for an iron truss bridge was taken out on April 24th, 1841, and, while this was not for a pin-connected truss, it possessed some of the features of the present practice in bridge design. As well as can be learned from the meager drawings and specifications then submitted for United States patents, this bridge may be described as follows: The top chord was a cast-iron arch, of channel section, widening in plan from the center towards the ends, built in sections "from 10 to 20 ft. long," and with the several parts united at the panel points by a kind of socket joint with a pin passing through both parts of the arch. The bottom chord was made of round rods, apparently connected at the foot of each vertical to a channel-shaped casting, which served as a transverse floor-beam, and secured by nuts inside this channel. The verticals were round iron rods, somewhat vaguely described as acting "as both ties and posts," and the diagonal system in each panel was made of rods, or "braces of cast iron," placed in pairs. According to the drawing, these diagonal rods were secured by nuts to the top chord and to the transverse floor-beam before described. Mr. Whipple built a number of these bridges, with spans ranging from 70 to 100 ft., over the Erie Canal, and of the fifty

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iron bridges erected in the United States prior to 1850 these structures constituted the large majority.

About 1852, Mr. Whipple designed and built several short and unimportant iron railway bridges upon the New York and Erie Railway; but though they successfully withstood severe tests and were in use for some months, they were removed owing to a panic caused by the failure of another iron bridge on the same line, of different design and proportions. But in 1852–53 he constructed a Whipple trapezoidal iron railway bridge, of 146-ft. clear span, upon the Rensselaer and Saratoga Railroad, near West Troy, N. Y. This is claimed to be the first iron bridge built with inclined end-posts, and the type was afterwards well known as the Whipple truss. It successfully withstood the gradual increase in the weight of rolling stock until 1882, when it was taken down and replaced by a double-track structure of modern design.

From an illustrated description of this bridge, written by Squire Whipple himself and published in *Engineering News* of April 7th, 1883, the following brief abstract is made : This bridge had parallel top and bottom chords, inclined end posts and a double intersection diagonal system, with the compression members made of cast-iron. The bottom chord was constructed of iron links of various diameters connecting over cast-iron trunnions forming a part of the cast post-shoe. The top chord was constructed of hollow cast-iron cylinders abutting on a round pin above the post; and this pin rested upon a semicircular groove in an extension of the post and also secured the tops of the diagonal members, which were provided with an eye for this purpose. The lower ends of the diagonals passed through the cast post-shoe and were secured there and adjusted by nuts. The posts were tapering hollow castings, in four sections, bolted together by flanged joints and trussed at the center to guard against buckling and to compensate for an opening left in the center of the post for the passage of the diagonals. While this bridge might be said to be pin-connected above, the bottom connection was made by the cast-iron trunnions before mentioned, which were from 7 to 9 ins. long and 3 ins. in vertical thickness, and rounded off at both ends to fit the links. This bridge was built upon a skew, with an angle of 44°, and it was originally submitted to a test of 150 tons of railway iron, distributed, and the passage of a 40-ton locomotive drawing a train of loaded freight cars covering the bridge. It was proportioned for a rolling load of 1 ton per foot only, and was probably the lightest iron railway bridge of like span ever constructed, for the metal in it is said to have weighed only 75 000 lbs., 43 000 lbs. of this being cast iron.

Early in 1847 Mr. Whipple published a brief but practical treatise upon bridge designing; and in 1869 he issued a continuation of this work, for which he set the type, made the woodcuts and printed off the sheets on a small hand-press at his own home. This original treatise is noticeable for the general soundness of his reasoning, and

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as being one of the very earliest attempts to handle the problems of bridge design and proportions upon well-digested scientific principles.

The full title of the book, as printed by H. H. Curtiss, of Utica, N. Y., was: "A Work upon Bridge Building, Consisting of Two Essays, the One Elementary and General, the Other giving Original Plans and Practical Details for Iron and Wooden Bridges." In this treatise Mr. Whipple pointed out and illustrated the fundamental law of framed structures and gave rules and formulas for determining the exact amount of stress brought upon the several members of a truss or bridge. He also discussed the relative economy of various arrangements of members, and, in fact, forecast much that is now actual practice. Mr. Whipple claimed no originality for the use of diagonal members between parallel chords; but he appears to have been the first to use pins in a truss of this character and to adopt the inclined end post, now in such common use.

In 1872 he published an enlarged edition of his book on bridge building, and a fourth edition was put out in 1883, including in this list of editions that of 1869, printed by himself. In 1866 he also published, at Albany, N. Y., a pamphlet entitled "The Doctrine of Central Forces, Illustrated Without the Use of the Calculus."

Among his other inventions was a lifting draw-bridge, designed by him in 1873 to meet various difficult conditions found in carrying streets across the Erie Canal. The chief feature of this bridge was a counter-balanced floor suspended to an elevated trussed structure spanning the canal. When space was required for the passage of boats this floor was lifted by gears and shafting, sufficiently to allow the boats to pass beneath. He built one of these lift bridges in Utica, over the Erie Canal, and it is believed to be still in use.

Personally, Mr. Whipple was of a retiring disposition, preferring the quiet of his study and mechanical experiments to the society of any except his few most intimate friends. To the latter he was always genial and entertaining. From his youth upward he had an inveterate aversion to violence of any kind, and especially as this violence was exhibited towards the dumb creation. For this reason he had always abstained from animal food, except such as is produced in the dairy, and he published, in 1847, a small pamphlet called the "Way to Happiness," in which he ably defended his position and his peculiar diet. He claimed that his practice was in the interest of good health, rational economy and sound morality, and the fact that he was the last of a family of ten children and lived to a sturdy old age himself would seem to indicate that he found some virtue in his peculiar methods and views.

Squire Whipple was elected an Honorary Member of the American Society of Civil Engineers, on May 6th, 1868. He died at his home in Albany, N. Y., on March 15th, 1888, in his eighty-fourth year, leaving a widow but no children.

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WILLIAM MILNOR ROBERTS, Past-President Am. Soc. C. E.*

DIED JULY 14TH, 1881.

William Milnor Roberts was of Welsh descent. His family were members of the Society of Friends, who came to America with William Penn. They resided in Philadelphia, and a grandfather and an uncle served as mayors of that city. His father's name was Thomas Pascal Roberts. His mother was Mary Louise Baker, and he, their eldest son, was born February 12th, 1810. He was educated at one of the schools of the Society of Friends, taking also two terms of a special course in mathematics under Professor Joseph Roberts. He then became a pupil of the first school established by the Franklin Institute, and there was taught architectural drawing by John Haviland, a well-known architect of that period.

In the spring of 1825, when hardly past his fifteenth birthday, he was made a member of the engineering corps engaged in the construction of the Union Canal of Pennsylvania. From that moment up to the day of his death, in July, 1881, his life was one of continued activity and of devotion to his profession. During those sixty-six years he saw, and in fact was a very important factor in, the wonderful development of public works, which was guided and directed by the civil engineers of America. When he began his engineering service in 1825, there was no railroad in operation in that country. The improvement of means of transportation by water was in active progress. The Erie Canal in New York, begun in 1817, was completed from Buffalo to the Hudson in October, 1825. The State of Pennsylvania had determined by legislative act to adopt and prosecute a canal system. The Chief Engineer who gave a position to William Milnor Roberts was Mr. Canvass White, who had been one of the engineers of the State of New York in the location and construction of the Erie Canal, and who first introduced the use of hydraulic cement in the construction of its masonry, having discovered in 1818 a stone in central New York from which it was made. The chief of the party in which the young man served was Mr. Sylvester Welch, a severe disciplinarian and accomplished engineer. The devotion and energy of the young assistant commended him both to Mr. White and Mr. Welch, and their opinion of him was shown by his rapid promotion to positions of responsibility. Mr. Roberts was really for a number of years both pupil of and assistant to Mr. Welch. In 1826 he was rodman with Mr. White in a survey made across the Allegheny Mountains with special reference to the construction of a macadamized road to connect the water transportation on the eastern and western slopes. In 1827 he was sent by Mr.

* Memoir prepared by John Bogart, M. Am. Soc. C. E.

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White as a young assistant upon the survey and construction of the Lehigh Canal, and here he made his first acquaintance with a railroad at the inclined planes at Mauch Chunk. He aided Mr. White in making improvements at those planes, and thus took part in the construction and operation of one of the earliest railroads in the United States. He was one of the passengers on the first trip of the first passenger car actually run on rails in this country.

He continued his service on the Lehigh River and Canal improvement until 1831, when he was appointed Senior Assistant Engineer under Mr. Sylvester Welch, Chief Engineer of the proposed Allegheny Portage Railroad. Surveys and plans for this road had been made in 1829 and 1830 by Mr. Moncure Robinson and by Col. Long, both including eleven planes, although not on the same location. Mr. Roberts suggested to Mr. Welch ten planes, all straight and with inclinations of from 7¼ to 10¼ per cent. He superintended the construction of the five on the eastern slope, and of three on the western slope. In 1832 he rode with Chief Engineer Welch across the country on horseback, and visited the inclined planes on the Morris Canal, and those on the Carbondale and Honesdale Railroad. He spent much time during that year also upon the Western Division of the Pennsylvania Canal, directing repairs made necessary by the great flood of February, 1832. On November 21st, 1833, the Allegheny Portage Railroad was finished, and a train of cars, drawn by horses on the levels, was passed over all the planes. It was opened to the public in 1834. Any persons who chose could put cars upon it, and haul them on the levels with their own horses. Mr. Roberts was given the general charge and management of the running of the road, and continued in this position until January, 1835, when he resigned and was appointed Chief Engineer of the Lancaster and Harrisburg Railroad.

This closed the first ten years of the engineering life of Mr. Roberts. At the age of twenty-five he had become the chief engineer of an important railroad enterprise presenting novel problems of location and construction, and with very little in the whole country to serve as model or guide. There must have been great promise in the young man to lead to his selection for such responsibilities. In fact, he had already developed the characteristics and qualifications which showed so clearly in his subsequent career. The history of that career will be concisely sketched directly.

Mr. Roberts did not have the education of the technical school; in fact, technical schools did not then exist. When he was young, there were no libraries of engineering information. The records of experience and the deductions from such records were in private notebooks, and in correspondence with the very limited number of active engineers. Specialism was not thought of. The varied questions which arose could not be referred—they must be answered at once.

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No men could succeed under such circumstances except those specially fitted for the task. Mr. Roberts was a very close and minute observer of physical facts related to engineering problems. He kept voluminous notes, and he had a very retentive memory. He had a special aptitude for design, and an appreciation of what was essential and non-essential in each particular case. He had much mechanical ability, and could draw without instruments a plan which would be surprisingly close to scale. He had in addition untiring energy and extreme devotion to the work of the moment. His diaries and letters and the statements of friends who were associated with him are full of the stories of his remarkable capacity for work. His frequent trips on horseback were for long distances and at great speed, and these never seemed to interfere with his ability for the immediate direction of affairs. In fact his life was of unceasing activity, not only in his younger days, but up to the very last. His physique enabled him to endure great fatigue. He was of moderate height, never corpulent, but of a wiry and muscular frame. In his younger days he was very fond of athletic sports, particularly of skating, in which he was very skillful. He was physically strong and personally brave. On one occasion, at the risk of his own life, he rescued an Indian mother and child who had fallen into Lake Erie from the deck of a steamer on which he was a passenger. In making a reconnaissance of a new region, riding with farmers or guides familiar with the trails, he would astonish them by the accuracy of his estimates of the distances traveled.

He permitted nothing to interfere with professional honesty to his duties. In two particular instances, one quite early in his charge of important work, the other after the war, he tendered his resignation of his position, when an attempt was made to influence politically his engineering appointments.

He was not a student in that sense of the word which is defined as one given to the study of books, but he was a student in the other sense, as one given to the acquisition of knowledge. His writings therefore were generally descriptions of actual observations and expressions of opinion as to the application of experience to new problems. He seldom developed mathematical theories, and he expressed at times doubts as to the reliance to be placed upon a too free use of theoretical formulas, particularly in cases where it could not be certainly known that the existing conditions were the same as those from which the formulas were deduced. The story of his professional life shows that he had a very large opportunity to study the problems of the movement and action of water. He was engaged in the improvement of many important rivers, among them the Monongahela, Ohio, Lehigh, Juniata, Kiskiminetas, Allegheny, Big Beaver, Little Beaver, Des Moines and Mississippi, and he had been during early life in the engineering corps in charge of the

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extensive canal works of Pennsylvania. In a letter referring to the discussions upon the vexed questions connected with the improvement of the outlet of the Mississippi he wrote:

"The nice mathematical theories of 'threads of water,' 'waves of translation,' 'radius vector,' etc., are well enough in their proper places. I do not object to them; but if a man has all of these and analogous things at his finger end, and has not practical experience in the actual operation on a large scale of water in rivers and canals, his judgment might easily be at fault when undertaking to plan river works or to criticise river plans. Gravity being the father of the whole thing, of course, he looks carefully after all his children. It is curious how simple things may be made mysterious."

This distrust of too free use of theory did not prevent him from being a welcome associate with many of the most distinguished theoreticians in the practice of engineering science. His wonderful fund of information, his knowledge of engineering facts, his lucid discussions, his unfailing good nature, and his excellently educated common sense, made him always a reliable and successful consulting or chief engineer, and a valuable member of many important commissions.

Mr. Roberts was Chief Engineer of the Lancaster and Harrisburg Railroad during 1835, 1836 and 1837. He located it with a maximum grade of 39.6 ft. per mile, grave doubts existing as to the capacity of a locomotive to haul loads on a steeper grade. At that time grades of 50 ft. per mile were operated in England, as inclines with rope traction. In 1836 he was made Chief Engineer of the Cumberland Valley Railroad, and finished the construction of both lines. During this year he visited Alexandria, Va., where he made a contract for two locomotives; New Castle, Del., one locomotive; Norris Works, Philadelphia, four locomotives; and Lowell, where, however, he could not secure any locomotives because of the extent of previous orders for other roads. In 1837 he built the combination lattice-truss bridge crossing the Susquehanna at Harrisburg, with a double-track railroad on top, and a double carriage-way and footpaths below. In 1838, 1839 and 1840, while still Consulting Engineer for the above-mentioned railroads, he took charge as Chief Engineer in the State service, of the extensions of the State canals of Pennsylvania, the work in progress extending from Erie, southerly, to Pittsburg, and thence still southerly on the Monongahela to the Virginia line, the latter being for the Monongahela Navigation Company. His duties carried him over wide tracts of country which he generally traversed on horseback, keeping three horses in service, and he generally rode from 30 to 40 miles a day, sometimes 50, and once 68 miles. During 1841 and 1842 he was engaged on the enlargement of the Welland Canal in Canada; 1843 and 1844, the Erie Canal of Pennsylvania; 1845 to 1848, Chief Engineer and Trustees' Agent for the Sandy and Beaver Canal in Ohio. During the latter year at the request of the Legislature of Pennsylvania he recommended a line for avoiding the Schuylkill inclined plane near Philadelphia,

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which was adopted and built substantially upon the route now used by the Pennsylvania Railroad. In 1849 Mr. Roberts declined the Chief Engineership of the first projected railroad in South America, to accept that of the Bellefontaine and Indiana Railroad where he remained until 1851. From 1852 to 1854 he was Chief Engineer of the Allegheny Valley Railroad, Consulting Engineer for the Atlantic and Mississippi Railroad, contractor for the construction of the Iron Mountain Railroad, and Chairman of a commission of three appointed by the Pennsylvania Legislature upon the reconstruction of the Allegheny Portage Railroad. This commission recommended the abandonment of the inclined planes and the construction of a railroad with maximum grades of 75 ft. per mile, which plan was adopted and the road built by the State. The Pennsylvania Railroad had previously built its line with grades of 95 ft. per mile. The two lines were within sight of each other, and the State railroad was soon after bought by the Pennsylvania.

This was the period of the battle of the railway gauges, and for awhile it seemed that the decision would be in favor of the wider, 6 ft. or more, as against the gauge of 4 ft. 8½ ins. While Chief Engineer of the Allegheny Railroad, which was to connect the Pennsylvania and New York systems, Mr. Roberts strongly advocated the narrow width. The officials of the company at first disagreed with him, and the ties were laid for the wide gauge, but when the rails were placed he succeeded in securing the adoption of what is now the standard gauge.

During 1855, 1856 and 1857 Mr. Roberts was engaged in the construction of the Keokuk, Des Moines and Minnesota Railroad and the Keokuk, Mt. Pleasant and Muscatine Railroad. He was also Consulting Engineer for the Pittsburg and Erie and for the Terre Haute, Vandalia and St. Louis roads. In December, 1857, he went to Brazil; in May, 1858, closed the contract for the construction of the Dom Pedro Segunda Railroad, and as a senior member of a firm built that road. It had thirteen tunnels in 9 miles, one, 7 200 ft. long; there were 3½ miles of tunnel in 9 miles of line. Mr. Roberts was in Brazil until the latter part of 1865, when he took charge of the surveys for the Atlantic and Great Western road, completing them in 1866. He was then appointed on the Commission which recommended the plans for the improvement of the Mississippi River at Keokuk, and in 1866 was appointed United States Civil Engineer in charge of the improvement of navigation of the Ohio River, continuing in this position till 1870. During this time, while on leave, he was in 1868 appointed, at the suggestion of Mr. Eads, Associate Chief Engineer to the St. Louis and Illinois Bridge Company, on the occasion of the visit of Mr. Eads to Europe on account of the serious condition of his health. Mr. Roberts continued in the service of the bridge company two years. He was constantly engaged in conjunction with the other engineers, Messrs.

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Flad and Pfeifer and Mr. McComas, the General Superintendent, in the personal direction of the sinking of the caissons of both the east and west piers, a work of novelty, delicacy and danger.

In the fall of 1869 Mr. Roberts became Chief Engineer of the Northern Pacific Railroad and continued in that position until his departure for Brazil in 1879. During this period he made extended surveys and examinations along the various parts of this transcontinental route, including trips over the western mountain passes. In addition to this service he made, in 1873, examinations and a report upon the Marquette and Mackinaw, and upon the Minneapolis and St. Louis Railroads. In 1874 he was made a member of the commission appointed by the President of the United States to examine and report upon the proper method of improving the mouth of the Mississippi. The other members were Gen. H. G. Wright, U. S. A.; Gen. B. S. Alexander, U. S. A.; Gen. C. B. Comstock, U. S. A.; Prof. Henry Mitchell, U. S. Coast Survey; T. E. Sickles, M. Am. Soc. C. E., and H. D. Whitcomb, M. Am. Soc. C. E. This commission visited the mouths of various European rivers. Their report was made January 14th, 1875. In 1874 he also reported upon the water-works of Pittsburg. In 1875 he was a member of a commission to examine and report upon the Water Supply of Philadelphia. He was also a member of the Advisory Commission of Engineers upon the construction of the South Pass jetties at the mouth of the Mississippi River. This board met first on September 2d, 1875. Its members were Gen. J. G. Barnard, U. S. A.; Gen. B. S. Alexander, U. S. A.; Sir Charles A. Hartley, W. Milnor Roberts, T. E. Sickles, M. Am. Soc. C. E.; Prof. Henry Mitchell, U. S. Coast Survey, and H. D. Whitcomb, M. Am. Soc. C. E. This board made its report November 20th, 1875, approving the works in progress.

In 1878 he was in Nova Scotia in connection with the location of the Nova Scotia, Nictaux and Atlantic Railway. In September of that year he was in Washington Territory, and crossed the Cascade Mountains on muleback. In December he concluded a contract for three years with the government of Brazil to act as engineer for that government on any public works, and on January 4th, 1879, left New York for Rio de Janeiro. After his arrival he made examinations of the port of Santos, of the Sao Francisco River, and of the ports of Pernambuco, Fortaleza, Maranhao, Vitoria and Caravellas. He also made a report on the water-works of Rio de Janeiro. On July 2d, 1881, he started to make an examination of the Rio das Velhas. He was compelled to suspend his journey on the 7th at a small settlement called Soledade in the province of Minas Geraes. His indisposition developed into typhoid fever, and he died at that place July 14th. He was buried in a neighboring cemetery, but was afterwards removed and buried in Woodlands Cemetery in Philadelphia, Pa.

Mr. Roberts was highly esteemed in Brazil, and especially by the engineers of that country. After his death the Engineers' Club of

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Rio de Janeiro sent to the American Society of Civil Engineers a communication expressing its personal and professional regard and appreciation of the services of Mr. Roberts, and its regret for his sudden death. This communication was accompanied by a bound volume of illustrations of the Dom Pedro Segunda Railroad, which Mr. Roberts began building in 1858, and which was the last railroad over which he traveled.

Mr. Roberts became a member of the American Society of Civil Engineers September 21st, 1870. He was elected Director in November, 1876; was Vice-President November, 1873, to 1876, and November, 1877, to 1878, and was made President of the Society November 6th, 1878. During the Exposition at Philadelphia in 1876 he was Chairman of the Finance Committee of the Centennial Commission of the Society, and devoted much time to the work of that commission. A number of papers and discussions by him have been published in the *Transactions*.

Mr. Roberts married June, 1837, Annie, daughter of Chief Justice John B. Gibson of Pennsylvania. She died in 1857. He married again in November, 1868, Adeline, daughter of Mr. Anthony Beelen, of Pittsburg, who survives him. He had six children by his first wife, and three by the second.

This memoir would be incomplete without a reference to the peculiar charm of the personality of Mr. Roberts. He was eminently social. He was always bright, hopeful, full of anticipation of good results from his earnest work. His character showed clear, straightforward, charming. In troubled seasons, and he had experiences of them, he was ever looking for the sunshine soon to come.

He was a remarkably rapid and voluminous writer, particularly enjoying correspondence with friends. Many of his letters, pages in extent, were written on railway trains, or in the evenings of days which had been so full of work as to tire out all his companions. These letters were witty, cheerful, full of kindly thoughts, and also full of most interesting details of the journey or the professional works which had been seen during the previous days.

He was a most genial companion, in fact he was the cheery life of the assemblages of which he was a part. He had a bright sense of humor, and a constant fund of stories of his varied experiences. He was always very dear to his friends.

His professional life began with the beginnings of American engineering. He took part in the earliest canal constructions, and in those improvements of river navigation which preceded the railway. In the fullness of his experience he aided in the most important development of the treatment of great rivers the world has ever known. He was one of those brave pioneers who built the first railroads of our country; and before the end of his life, he was the chief engineer of a great transcontinental line. With all his achievements and experiences, he was always unassuming, genial, courteous, "a true and kindly gentleman, pure and modest."

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JOB ABBOTT, M. Am. Soc. C. E.*

DIED AUGUST 18TH, 1896.

Job Abbott was born at Andover, Mass., August 23d, 1845. He attended the district schools and Phillips Academy before going to Harvard, where he studied engineering in the Lawrence Scientific School. After graduating in 1864, he was connected for a time with the Manchester Locomotive Works, but his active engineering work began when he was appointed Assistant Engineer on the Glen Cove branch of the Long Island Railroad. This engagement was a short one, and on its termination he went to the Pittsburg, Ft. Wayne and Chicago Railway in a similar capacity. While on this line he became interested in the city of Canton, O., a large part of which he laid out, and there he had an office as civil and mining engineer and patent expert from 1866 to 1872. During this time he studied law and was admitted to the Ohio bar.

While practicing patent law in Canton he was retained on certain matters by the Wrought Iron Bridge Company, and became so interested in these affairs that he dropped his legal practice to take up engineering again. He was Vice-President and Chief Engineer of the Wrought Iron Bridge Company from 1872 to 1880, and remained a Director until his death. During this time he built some bridges in Canada, and believing the Canadian business to have a bright future, he helped organize the Toronto Bridge Company, of which he was President and Chief Engineer from 1880 to 1884. The business in Toronto soon outgrew the capacity of the shops there, and Mr. Abbott accordingly helped to organize the Dominion Bridge Company of Montreal, of which he was President and Chief Engineer from 1884 to 1888, and President until 1890. The shops of both these companies were designed, erected and equipped under his direction, and during his connection with them he built some of the largest bridges in Canada. In 1884 and 1885 he designed and had charge of the 477-ft. steel cantilever bridge at St. John, N. B,; in 1887, a steel bridge 2 020 ft. long on the Fredericton Railway; in 1888, the superstructure of the Canadian Pacific bridge, 2 440 ft. long, at Sault Ste. Marie; in 1889, the Grand Narrows steel bridge, 1 715 ft. long, on the Dominion Government Railway in Cape Breton, N. S. In 1886 and 1887 he also had charge of building the superstructure of the Lachine Bridge, 3 660 ft. long, over the St. Lawrence River on the Canadian Pacific line.

In 1889 he found he was working too hard, and gave up his Montreal office in order to remove to New York, where he established an office as consulting engineer, which he maintained until April, 1896.

* Memoir prepared from papers on file in the House of the Society.

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In 1889 and 1890 he designed and had charge of the Ohio River Bridge at Wheeling, W. Va., and was appointed Chief Engineer of the Wheeling Bridge and Terminal Railway Company. The work done by this company was very heavy, the bridge alone being a double-track structure 2 097 ft. long with a 525-ft. channel span.

After the completion of the Wheeling works, Mr. Abbott was retained as Consulting Engineer for the Bangor and Aroostook Railroad in Maine, the longest line built in New England since the Central Massachusetts was finished. While engaged in Maine in work for this road, he was taken ill but kept at his labors for about a year, when he went to Andover for health and rest. He was too late, however, and after a trying sickness died August 18th, leaving a widow.

Mr. Abbott was elected a Member of the American Society of Civil Engineers, April 1st, 1891.

WILLIAM ALBERT ALLEN, M. Am. Soc. C. E.*

DIED MARCH 21st, 1896.

William Albert Allen was born at Bath, Me., October 18th, 1852. He graduated in 1874 from the Maine State College, where he studied civil engineering, and a year later entered the service of the Maine Central Railroad Company as assistant engineer. At that time the policy the management of this company was rapidly adopting aimed at the construction of a modern, high-class road-bed with good iron bridges, and Mr. Allen's attainments made him a valuable man for such work. In April, 1877, he was made Civil Engineer of the road, and in January, 1885, was promoted to the position of Chief Engineer, with the control and responsibility that the title implies. In 1887 and 1888 he was also Chief Engineer of the Portland Union Railway Station Company, and held the office from the commencement of the work until the completion of the station, train-shed and approaches.

Mr. Allen lost his life on March 21st, 1896, while inspecting work that was being done on the railway bridge between Auburn and Lewiston. He was leaning out from the steps of a car to observe the progress of the work, when he lost his hold or was struck by a telegraph pole. He fell from the car to the bridge, from which he dropped into the river above the falls at this place. He was carried over the falls and his body was not recovered until some weeks later. He was a widower and left no children.

Mr. Allen was elected a Member of the American Society of Civil Engineers, May 6th, 1891.

* Memoir prepared from information furnished by Mr. H. C. Robinson, Assistant Engineer Maine Central Railroad, and from papers on file at the House of the Society.

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JAMES BARNES, M. Am. Soc. C. E.*

DIED FEBRUARY 12TH, 1869.

General James Barnes, a prominent engineer and railroad contractor before the Civil War, and the forty-fourth Member of the American Society of Civil Engineers, was a native of Massachusetts. He was graduated in 1829 from the United States Military Academy at West Point, ranking five in a class that included, among others, Robert E. Lee and Joseph E. Johnston, against whom he was destined to carry out later those principles of warfare which they studied as classmates. After graduation he was appointed to the artillery branch, and served for about a year at the Academy as an Assistant Teacher. From 1830 to 1833 he was on duty at Fort McHenry, Md.; Charleston, S. C., and Fort Monroe, Va. Then he served as an Assistant Instructor of Infantry Tactics at West Point until August, 1836, when he resigned to enter upon a prosperous career as engineer and contractor.

From 1836 to 1842 he was Assistant Engineer of the Western Railroad, extending from Worcester, Mass., to Albany, N. Y., and now a part of the Boston and Albany Railroad, and from 1842 to 1848 he was its Chief Engineer and Superintendent. Then he went south, and from 1842 to 1848 was Chief Engineer and Superintendent of the Seaboard and Roanoke Railroad from Norfolk, Va., to Weldon, N. C. It was during this time that he became interested in contracting as a member of the firm of Phelps, Mattoon and Barnes, of Springfield, Mass., builders of the Watertown and Rome Railroad. This was a very successful contract, and led to his engaging in this business for a number of years, during which he built the Sackett's Harbor and Ellisburg Railroad, part of the Buffalo, Corning and New York Railroad, the Terre Haute, Alton and St. Louis Railroad and the Potsdam and Watertown Railroad. During 1853 and 1854, he resided in Russia, where he was retained by the Russian Government as Consulting Engineer of the railway from St. Petersburg to Moscow.

On the outbreak of the Civil War, General Barnes at once volunteered, and became Colonel of the 18th Massachusetts Infantry in July, 1861. He served with distinction in the operations of the Army of the Potomac, and was in command of a division at the battle of Gettysburg, in which he was wounded. During the latter part of 1863 he was in command of the defences of Norfolk and Portsmouth, Va., and then spent six months on court martial duty. He was in

* Memoir prepared from an article in the "Biographical Register of the Graduates of the U. S. Military Academy," by Gen. G. W. Cullom, and from information furnished by John S. Barnes, Esq.

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command of the military prison at Point Lookout, Md., and of the St. Mary's District until the middle of 1865, when he was placed on waiting orders. On March 13th, 1865, he was brevetted Major-General of Volunteers for meritorious services during the Rebellion, and on January 15th, 1866, he was mustered out of service.

In 1868 he was a member of a special United States Commission to examine and report on the road and telegraph line of the Union Pacific Railroad Company.

He was a hard student all his life, a good Greek and Latin scholar, and able to speak and write German, French, Spanish and Russian fluently. His business methods were careful and exact, and his personal attributes endeared him to all who knew him intimately.

General Barnes died at Springfield, Mass., February 12th, 1869, at the age of sixty-three years. He was elected a Member of the American Society of Civil Engineers March 13th, 1853.

HENRY ISAAC BLISS, M. Am. Soc. C. E.*

DIED JULY 10TH, 1896.

Henry Isaac Bliss was born at Hartford, Conn., January 8th, 1830. He entered Yale College in 1848, but previous to that time had studied engineering, and had some practical experience. After graduating from the college in 1853, he went at once to Wisconsin and became connected with the construction of the Milwaukee and Fond du Lac Railroad, afterward the La Crosse and Milwaukee Railroad, and now part of the Chicago, Milwaukee and St. Paul system. In the fall of 1854 he returned to Connecticut, and was engaged near Middletown on the construction of the Boston and New York Air Line Railroad until work was suspended in 1855. Then he returned to Wisconsin and made a survey of the La Crosse and Milwaukee Railroad from La Crosse to the Wisconsin River.

In the spring of 1856 he settled in La Crosse, which remained his home up to the time of his death. He at once became engaged in engineering and the real estate business, and made a number of railway surveys. From 1860 to 1884 he was City Engineer of La Crosse, and during this time laid out much of the city as it now stands, and built the municipal water-works plant. He was elected a Member of the American Society of Civil Engineers on September 5th, 1883. Mr. Bliss was in feeble health for some time previous to his death on July 10th, 1896. He leaves a widow and married daughter.

* Memoir prepared from papers on file at the House of the Society.

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HENRY D. BLUNDEN, M. Am. Soc. C. E.*

DIED JANUARY 7TH, 1889.

Henry D. Blunden was born at Poling, England, April 8th, 1849. He was educated at Brighton and London, England, and at the age of 16 years was articled to John Lawson, C. E., for a term of four years. During 1869 and 1870 he was employed as Contractor's Engineer on the construction of the Mansfield and Southwell Branch of the Midland Railway, England. From August, 1870, to July, 1872, he was Assistant Engineer on the Kansas Pacific Railway, making preliminary surveys and locating new towns; from July, 1872, to July, 1873, Assistant Engineer on the Leavenworth, Lawrence and Galveston Railway, making preliminary surveys and building machine shops; from July, 1873, to 1882, Assistant Engineer on the New York, Lake Erie and Western Road. During 1882 and 1883 he was Road Master on the Eastern Division of the Erie Road, and on September 13th, 1883, he was appointed Engineer of Maintenance of Way of the same road, holding this position until 1886. During 1887 and 1888 he was Assistant Engineer at the Union Bridge Company's shops at Athens, Pa., and was also during this time Superintendent of Bridges on the Pennsylvania and New York Railway, and the Geneva, Ithaca and Sayre Railway. His death occurred January 7th, 1889.

Mr. Blunden was elected a Junior of the American Society of Civil Engineers on January 5th, 1876, and a Member on February 4th, 1880.

ROBERT BRIGGS, M. Am. Soc. C. E.†

DIED JULY 24TH, 1882.

Robert Briggs was a native of Boston, Mass., being born in that city on June 18th, 1822. His early education was obtained in the Boston public schools, and, while there, his aptitude for mathematics, which was afterwards one of his marked characteristics, was noticeable. His engineering training began at the age of seventeen, in the office of Alexander Parris, a local engineer and architect, where he remained for several years. His subsequent career was remarkable for the wide range of engagements it embraced. He designed and erected industrial plants of great size, and afterwards operated them, turning out a

* Memoir prepared by W. B. Coffin, M. Am. Soc. C. E.

† Memoir prepared from information furnished by Henry G. Morris, M. Am. Soc. C. E., and papers on file at the House of the Society.

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great variety of products, ranging from tubes to pumping engines. His practice as civil engineer was equally broad. In addition to the professional attainments which enabled him to carry on these works with marked success, he was a clear and ready writer on technical and scientific subjects, the author of several papers before engineering societies, and the editor, for a time, of a well-known scientific journal. The surprising thing is, that, with such versatility and such success in practical charge of important work, he was never a strong man, and was frequently invalided and prevented from putting into full play the unusual gifts he possessed.

From 1844 to 1847 Mr. Briggs was not engaged in engineering work, but in the latter year he worked for a few months on a railway line in Massachusetts. Then he became Constructing Engineer of the Glendon Rolling Mill, an important establishment then in course of construction at East Boston, and thus acquired his first taste of a branch of work in which he was destined to achieve a marked success in later years. When the mill was finished, he opened an office in Boston as a Consulting Engineer. This proved unprofitable, and he soon entered the service of Walworth & Nason, of Boston, with whom the application of steam to the warming of buildings in this country may be said to have originated. Mr. Briggs took a deep interest in this subject and in ventilation, and wrote several valuable papers on the subject, one of which was published in Volume X of *Transactions* of this Society. He took charge of the construction of the tube works of Walworth & Nason, and, when it was finished, served as its Superintendent for some time.

His restless nature compelled him to seek another field of work before long, and in 1852 he became Superintending Engineer of the firm of Bird & Weld, later the Phœnix Iron Works, at Trenton, N. J. After a year in this place he moved to Mount Savage, Md., to become Superintendent of a rolling mill there, for a period of six months. Then he went to Troy, N. Y., as Superintendent of the Rensselaer Rolling Mill, a position he held for a year. After this experience in mechanical engineering, he turned his attention to civil engineering and architecture again, as Assistant Engineer under General M. C. Meigs. He was engaged on the construction of the Washington aqueduct, the dome of the Capitol at Washington, and the heating and ventilating systems of the halls of Congress. In connection with the last work he made an investigation into the proportions of rotary fans, the results of which were subsequently embodied in a paper presented to the Institution of Civil Engineers, and given the Watt medal and Telford premium of that society.

In 1857 he became a member of the firm of Nason, Dodge & Briggs, of New York, but this connection was brief, for in 1860 he went to Philadelphia as Superintendent and Engineer of the Pascal Iron Works

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of Morris, Tasker & Company. This position was the longest he ever held. The works were then comparatively disorganized, and his first labor was to bring them into a good condition for the manufacture of pipe, fittings, pipe-cutting machinery, gas works appliances, and the other specialties of the company. When this was done, he systematized all these products, placing this part of the business on an excellent footing, and designed and built new additions to the works.

One of the novel features introduced by Mr. Briggs in the construction of gas holders was the flat-top holder without interior trussing. The first structure of this type was built by him for Lewiston, Me. This method of construction, now recognized as being correct, was strenuously opposed and ridiculed at the time by many engineers and prominent builders of gas holders.

In 1866 he visited Europe for the company and made the acquaintance of many foreign engineers, especially in England, where his wide knowledge of American engineering works made him a welcome visitor in professional circles. When his connection with the Pascal Iron Works was terminated in 1869, he revisited England and increased his knowledge of English practice and widened the circle of friends he made on his previous stay in that country.

Early in 1871, he became Engineer and Superintendent of the Southwark Foundry, then owned by Henry G. Morris, M. Am. Soc. C. E. Here he designed and built many large pieces of machinery, including a pumping engine for the city of Lowell, Mass., which had a remarkably good duty for that time, sugar machinery, gas apparatus, blast furnace fittings, boilers, engines and similar heavy work. He designed and built a large foundry during his connection with Mr. Morris, and a 30-ton traveling power crane. He remained in this position until the closure of the works in 1875 on account of the disturbance of the iron market at that time.

After a long illness which attacked him in 1875, Mr. Briggs made a short trip to England, returning to this country the next year to become editor of the *Journal* of the Franklin Institute, a post for which his wide experience and readiness as a writer made him particularly fitted. His work was well performed, but he wearied of it in a few years, and in 1878 opened an office in Philadelphia as a consulting engineer. This was not very profitable to him, but gave him time to prepare a number of valuable articles on technical subjects.

In 1880, he became Consulting Assistant to Col. William Ludlow, who was then in charge of river and harbor improvement in the vicinity of Philadelphia. He also retained his private practice, and was particularly interested in matters pertaining to heating and ventilating.

His health, never strong, began to fail noticeably in a short time, and late in 1881, after a brief visit to England, symptoms of paralysis appeared. He continued to work until April, 1882, when his physician

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made him desist and go to his mother's home in Dedham, Mass. His vitality was too exhausted, however, for him to recover, and he died there of paralysis on July 24, 1882, after a long and painful sickness.

The professional attainments of Mr. Briggs have been described as follows by Mr. Henry R. Towne, an intimate acquaintance.

"One of his most notable traits was the comprehensive scope of his knowledge, which covered almost the entire field of engineering, both civil and mechanical, and included much also of metallurgy, chemistry, architecture and the applied sciences. On almost any topic under these many heads he could discourse as a master with a minuteness and familiarity astonishing to any but those who knew what an extraordinary range was covered by his own personal experience in connection with mechanical and industrial operations, and who knew also how far these were supplemented by professional study and reading, continued uninterruptedly during the forty years of his business life. Added to these were advantages of a good early education and exceptional aptitude for mathematics, in which he excelled, and a very retentive memory."

Personally, Mr. Briggs had an even, quiet disposition, and a remarkable faculty for making and retaining friendships.

He was elected a Member of the American Society of Civil Engineers on October 19th, 1870.

ROBERT LINAH COBB, M. Am. Soc. C. E.*

DIED JUNE 2D, 1895.

Robert Linah Cobb was born at Cumberland Iron Works, Tenn., on March 5th, 1840. He was educated in the local schools and at Stewart College, now the Southwestern University, and after the completion of his studies, became a rodman on the Memphis, Clarksville and Louisville Railroad, now part of the Louisville and Nashville system. He soon became Assistant Engineer on this road, and in 1859 and 1860 was City Engineer of Clarksville, Tenn.

On the outbreak of the war he offered his services to the Confederacy and was assigned to the ordnance department. He was ordered to Fort Donelson, and, after the surrender of that post, reported to General Johnson at Nashville. Captain Cobb was then assigned to the engineering department and ordered to Corinth. The service he rendered as a military engineer was characteristic of the man, always faithful, painstaking, tireless and courageous. He was never taken prisoner, and served continuously until the close of the war.

* Memoir prepared by E. C. Lewis, M. Am. Soc. C. E.

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After the completion of this experience, he became Division Engineer of the Memphis, Clarksville and Louisville Railroad, and in 1867 was appointed Chief Engineer of the Winchester and Alabama Railroad, now part of the Louisville and Nashville system. Two years later he became Assistant Engineer of the Memphis and Ohio Railroad, from which he went in 1869 as Division Engineer to the Memphis and Little Rock Railroad, where he remained until 1872, serving part of the time as Acting Chief Engineer. In 1873 he conducted mining operations at Kellogg, Ark., and later in Mexico, but returned to railroad work in 1876 as a member of the engineering staff of the Little Rock and Fort Smith Railway. In 1881 and 1882 he was Chief Engineer of the Texas and St. Louis Railway, now the St. Louis, Arkansas and Texas. The next four years were spent in manufacturing machinery at Little Rock, Ark. In 1886 Captain Cobb was appointed Chief Engineer of the Indiana, Alabama and Texas Railroad, and upon the absorption of that line in 1887 by the Louisville and Nashville Railroad Company, he was retained by the latter company as Chief Engineer of construction, a position he held at the time of his election as a Member of the American Society of Civil Engineers on January 2d, 1890. In this year he was appointed Chief Engineer of the Clarksville Mineral Railroad, and in 1892 went to Ohio as the Chief Engineer of the Ohio Southern Railroad. Upon the completion of his work on this line he removed his headquarters to Cleveland, where, in the winter of 1894, he had an attack of grip from which he never recovered. He returned to the South in May, 1895, and died on June 2d at his early home, Clarksville.

His life's work speaks for him professionally. Personally he was at once a loyal friend and a chivalrous foe. He was married in 1877, and was left a widower ten years later. One son survives him.

ZERAH COLBURN, M. Am. Soc. C. E.*

DIED APRIL 26TH, 1870.

More than a generation has passed away since the brilliant career of Zerah Colburn closed prematurely in the snow-covered orchard at Belmont, Mass. It is somewhat difficult, after the lapse of six and twenty years to recall with accuracy or consecutiveness the details of his feverishly busy life, and still more difficult to find authentic records,

* Memoir prepared by James Dredge, Esq.

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which are more valuable than personal reminiscence. On the other hand, viewed in the perspective given by time, the errors that wrecked his life assume their proper proportions, and have ceased to tarnish the great qualities that made Zerah Colburn one of the famous engineers of his period. The curious autobiography of the earlier Zerah Colburn, published in 1833, throws some light upon the status of his family. With two exceptions it was of very ordinary stuff; the paternal grandfather was a needy unsuccessful farmer in New England; his wife a woman of strong intellect and sound judgment, who kept the home together and brought up a family, while he was seeking fortune in Europe by the exhibition of the extraordinary powers of calculation possessed by one of his children, the only member of the family who was fit for better things than the routine of farm labor. This was the first Zerah Colburn, who apparently gained more celebrity in Europe than in his native country. It is remarkable that the wonderful gift he enjoyed brought him so poor a result; in quite early life he became a minister of some dissenting denomination, and we hear no more of his calculating powers. The second Zerah Colburn, the subject of this sketch, was the son of one of the calculating Zerah's brothers, who lived and died unknown. The younger Zerah was born in 1832, shortly after his uncle had become a preacher, and when the honors of his calculating prowess were still thick upon him. Besides the name, Zerah Colburn inherited a vein of genius from his uncle, though of a different nature and tending to a more useful purpose. The farm which was his birthplace was in Saratoga, N. Y. Here his earliest years were passed, almost devoid of education, and filled with monotonous labor. But very soon—probably before he was ten years old—his extraordinary force of character and precocious development had revealed to him that the possibilities of a great future lay before him. Whence came the strong bent towards mechanics is as deep a mystery as the source from which his uncle's mathematical powers were derived, but it forced him onward, first to some local cotton mill, and afterward to the shops of the Concord Railroad, were the then superintendent, Mr. Minot, gave him his first chance, by which he profited so well that he speedily became a person of some importance on the railway.

Colburn must have stayed some years with Mr. Minot, probably three or four, but he left him in order to enter the Tredegar Locomotive Works at Richmond, Va. I believe, indeed, that there was an intermediate stage in his career, though a rather brief one. This was a short stay at some locomotive works in Boston, during which time he so far qualified himself, and so gained the confidence of capitalists, that before he was twenty-one he was a managing partner in the Tredegar Works. That he should have traveled so fast on the road to success was indeed marvelous. He was without education, except

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what was self-acquired; he had no influence and only self-made friends, and the lack of steadfastness in purpose, which proved in the end his ruin, was already making itself felt. But he possessed unlimited belief in himself, his power of absorbing and retaining knowledge appeared unbounded, he possessed an apparently inexhaustible store of physical strength, and he owned these qualities to such a degree that almost to the close of his life they more than counterbalanced the disadvantages of circumstance and character.

With the termination of his connection with the Richmond Locomotive Works, Colburn's practical career came to an end, and when he was little more than two and twenty he had entered on the new path which he followed more or less fitfully to the end of his life. This was the career of technical journalism. In 1851 the engineering profession had no press worthy of the name, and Colburn realized that therein there was a great future for him. He had already made successful experiments in this direction; when he was only fourteen or fifteen years of age he had written and published (finding the means somehow, as he always contrived to do) a monthly magazine devoted to engineering subjects; it was but a small, poor thing, but nevertheless it contained the germs of the marvelous engineering press of today. This publication, under the name of *Monthly Mechanical Tracts*, had but a short existence, and had become extinct before he left Mr. Minot, about 1849. Afterward he contributed to the *American Railway Times* of Boston, and later to the *American Railway Journal*, of which he was for some time assistant editor. But these efforts were only experimental, and in 1854 he commenced an engineering journal on his own account under the title of the *Railroad Advocate*. This, in the course of a short time, he developed into a relatively great success, so much was it impressed with the genius of its powerful editor. But it was impossible for Colburn to work steadily for any length of time; he had made his mark as an original thinker, an able engineer and a powerful journalist, but, having achieved these things, he tired of his enterprise, and sold it to Alexander L. Holley, who had not then risen into fame, and who had the fancy of owning and editing an engineering paper. The venture was not profitable, except for Colburn, and the journal dragged on a languishing existence for about 18 months, during which Colburn was a frequent contributor, at the same time attempting the utterly uncongenial and unsuccessful attempt to establish himself as a commercial engineer.

The summer of 1857 saw him in Europe, and on his return he bought back an interest in the *Railroad Advocate*, changed the name to the *Railway Engineer*, and conducted it with Holley for a year longer, when it died. He had returned to Europe in the fall of 1857 on a special mission, the work and pleasures of which he shared with Holley; at this time Colburn was not twenty-six years of age, Holley

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being a little his senior. These two brilliant young men visited Europe at the request of several leading American railroads with the purpose of reporting on English railway practice. The visit proved to be a turning point in Colburn's life; it terminated—with a few fitful exceptions—his residence in the United States, and made England his future home. Returning to America in the fall of 1857, he and Holley, in an incredibly short time, produced what was for a long time a standard book on European railways. In 1858 Colburn was back in London, and during the summer of that year he became editor of *The Engineer*, of London, a journal that had been started and feebly conducted for a few years. Colburn's energy, skill and brilliant qualities soon impressed themselves on the paper and raised it into a strong position, but he tired of the work, and returning to America in 1860, started a paper in Philadelphia on his own account, and bearing the same title, *The Engineer*. After five months this was abandoned, and the next year he returned to London, and was gladly received again as editor of *The Engineer*. This position he retained longer than any other during his life, for he did not finally quit his active work as its editor till 1864.

Then followed a period of general engineering practice and the preparation for the final venture of his life, the establishment of *Engineering*. The first number of this journal appeared in the beginning of January, 1866, and during the next sixteen months almost every page bore the stamp of his energy and genius. With the opening of the Paris International Exposition of 1867 he once more, and for the last time, fell away from the path of usefulness and work, and a record of the next four years would be but a melancholy story, relieved by occasional flashes of light, sufficiently brilliant, however, to prove that his great powers remained unaffected to the last. The facts may be stated now after so many years. Colburn's besetting weakness, or rather his incurable disease, was one before which many full of talent, like De Quincy and Edgar Allan Poe, had fallen before his time.

During the brief years into which Colburn's active career must be compressed, he achieved a high and lasting reputation, and it may in all justice be claimed for him that he was the creator of engineering journalism. It is believed that the following list of his life's literary work is a fairly complete one; it is compiled to a large extent from his own memoranda:

1847.—Wrote *Monthly Mechanical Tracts*, published at Lowell.
1848.—Wrote *The Locomotive Engine*, published in Boston.
1851-53.—Was contributor to the Boston *American Railway Times*.
1852.—Was contributor to the *Carpet Bag*.
1853.—Became assistant editor of the *American Railroad Journal*.
1854.—Started, as editor and proprietor, the *Railroad Advocate*.

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1856.—Sold the *Railroad Advocate* to A. L. Holley and attempted several things, contributing at the same time to the *Railroad Advocate*.

1857.—Visited Europe, and on his return bought an interest in the *Railroad Advocate*.

1857.—Second visit to Europe; this time with Holley to report on European railway practice.

1858.—Published, with Holley, "The Permanent Way and Coal-Burning Locomotive Boilers of European Railways."

1858.—Wrote the American section of D. K. Clark's book on "The Recent Practice in the Locomotive Engine."

1858.—Became editor of *The Engineer*, London.

1860.—Published an essay on "Steam Boiler Explosions" (John Weale & Sons).

1860.—Started, as editor and proprietor, *The Engineer*, Philadelphia.

1861.—Became editor of *The Engineer*, London, for the second time.

1863.—Wrote "An Enquiry into the Nature of Heat," published by Spon.

1863.—Wrote paper for the Institution of Civil Engineers "On American Iron Bridges."

1863.—Wrote paper for the London Society of Engineers "On the Relation Between the Safe Load and the Ultimate Strength of Iron."

1864.—Published "Gas Works of London" (Spon).

1864.—Wrote a part of "Locomotive Engineering." (This was published by Collins & Co., of Glasgow, but was not finished by Colburn.)

1864.—Resigned the editorship of *The Engineer*.

1864.—Read a paper, "A Description of the Harrison Steam Boiler," before the Institution of Mechanical Engineers.

1865.—During this year Colburn was a contributor to *The Engineer*, and he wrote and read the following papers: "On Certain Methods of Treating Cast Iron in the Foundry" (Society of Engineers); "On the Ginning of Cotton," and "On the Manufacture of Encaustic Tiles, etc., by Machinery" (Society of Arts).

1866.—Founded London *Engineering*.

1869.—Read papers "On American Locomotives and Rolling Stock" before the Institution of Civil Engineers; and a second, "On Anglo-French Communications," before the Society of Arts.

Zerah Colburn was a member of several institutions, among others of the American Society of Civil Engineers, of the London Institution of Civil Engineers, of the Institution of Mechanical Engineers, and of the Iron and Steel Institute; he was also a President of the London Society of Engineers. He was elected a Member of the American Society of Civil Engineers on January 5th, 1855.

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ADDISON CONNOR, M. Am. Soc. C. E.*

DIED JANUARY 4TH, 1891.

Addison Connor was born in New York City on April 2d, 1847. He was a graduate of Tufts College and of the Massachusetts Institute of Technology, and later in his life was given the degree of M. A. by the former institution. From 1871 to 1873 he served as assistant in the office of a Boston engineer, and then became transitman on the works for an additional water supply for the city of Boston. This position he held for seven years, when he became an assistant to Clemens Herschel, M. Am. Soc. C. E., and was engaged on surveys and tests of hydraulic machinery in the neighborhood of Holyoke, Mass.

In 1881, he became Resident and Assistant Engineer on the Northern Pacific Railroad, and built the company's dock at Superior, Wis., a structure 1 000 ft. long and 166 ft. wide. He was then transferred to the Missoula Division in Western Montana, and afterwards to the Cascade Division. When work on this railroad was suddenly suspended in 1884, he came east and entered the Department of Public Works of New York City as Assistant Engineer. Two years later he left this position to go to Plattsmouth, Neb., where he was employed by George S. Morison, M. Am. Soc. C. E., on surveys of the Missouri River at that place. He spent about seven months in 1886 and 1887 at Nebraska City, being engaged on preliminary surveys for the rectification of the river and the location of the bridge subsequently built there. He left Nebraska City early in June, 1877, to become one of the assistant engineers on the Cairo Bridge. Shortly before the completion of this bridge he was transferred to St. Louis, where he was engaged on some city work in connection with the new line of the St. Louis, Keokuk and Northwestern Railroad into that city. On the completion of his work there, he left Mr. Morison's employ, and in 1890 was appointed Assistant Engineer in the Department of Docks of New York City, where he remained until his death, which occurred January 4th, 1891.

Fidelity to duty was one of Mr. Connor's conspicuous traits. Modest and unassuming, although well equipped in his profession, the uniform reliability of his engineering work in various branches had gained for him the implicit confidence of those who employed him and who will remember kindly his able and conscientious collaboration.

Mr. Connor was elected a Member of the American Society of Civil Engineers on January 5th, 1887. He was married in 1876 to Mary E. Childs, of Framingham, Mass., who survived him.

* Memoir prepared from information furnished by George S. Morison, A. Fteley and Fred. Brooks, Members Am. Soc. C. E., and from papers on file at the House of the Society.

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ECKLEY BRINTON COXE, M. Am. Soc. C. E.*

DIED MAY 13TH, 1895.

Eckley Brinton Coxe, born at Philadelphia on June 4th, 1839, came of a family long identified with American history. In the last decade of the seventeenth century, one of his ancestors, Dr. Daniel Coxe, of London, purchased a patent granted originally by Charles I and covering the territory between the 31st and 36th parallels, and from the Atlantic to the Pacific, which was exchanged by his grandchildren for 100 000 acres in New York. He also had large holdings in New Jersey, and did much to build up trade in that colony. Dr. Coxe spent most of his life in London, but his son, Col. Daniel Coxe, lived in America from 1700 until his death in 1739. He took a prominent part in local political matters, and was a Judge of the New Jersey Supreme Court. His son, William Coxe, was a merchant in Burlington, N. J. The son of William Coxe, Tench Coxe, was a well-known figure in public affairs during the early history of the United States. He held several national offices and devoted much time to encouraging the growth of cotton, and it was to his recognition of the value of the Pennsylvania coal beds that the present mercantile position of the Coxe family is largely due. He formed a partnership with some acquaintances and acquired some of this coal land. His son, Judge Charles Sidney Coxe, bent most of his energies to extending and uniting this coal property, and when he died, in 1879, the great estate was in an excellent condition, financially and legally.

Just as Judge Coxe had his life work laid out by the affairs left him by his father, so did his children find their labors were outlined by the family coal properties. Eckley B. Coxe, one of these sons, soon became prominent as their technical administrator, and his engineering knowledge and experience made him one of the foremost mining engineers of his time.

He graduated from the University of Pennsylvania in 1858, and subsequently took a post-graduate scientific course there. His early knowledge of practical engineering matters was acquired during the summers spent on the family coal lands, where he was often a member of surveying parties. He thus became acquainted with the details of mining and preparing coal, and also of the topographical and geological features of the great property, so that when he began his technical studies, their practical import was apparent to him from the outset.

His progress in the European schools was excellent. At the École Nationale des Mines in Paris, where he remained for two years, he is

* Memoir prepared from papers on file at the House of the Society.

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still remembered, and his interest in that institution throughout his life was marked by many donations to it. At Freiberg he became well acquainted with Prof. Julius Weisbach, and published a translation of a portion of the latter's text-book on mechanics, which has long been recognized as an engineering classic.

In 1864, he returned to the United States. He was then twenty-five years of age and possessed of a wide theoretical knowledge of the principles of mining engineering. The great coal properties of the family estate were then worked in part under leases held by large operators, and Mr. Coxe quietly went about to acquire these leases, so that the entire output of the property would be under the control of its owners. Such a task was naturally a difficult one, but it was gradually accomplished. The Cross Creek Coal Company, of which Mr. Coxe was president, was organized to manage the mining operations. The Delaware, Susquehanna and Schuylkill Railroad Company, owned by the members of the Cross Creek Company, was organized to give the collieries of the estate an independent railway connection with the Pennsylvania, New Jersey Central, Reading and Lehigh Valley systems, so that the transportation of coal to the seaboard was not confined to a single line. A selling agency, Coxe Brothers & Company, was incorporated to market the coal, and built docks at Perth Amboy, Buffalo, Milwaukee and Chicago. The Coxe Iron Manufacturing Company was organized to carry out the construction and repair of the mining and railway machinery of the allied companies, and also took large outside contracts for machinery.

As the president of all these companies Mr. Coxe was naturally a busy man, yet he found time to give personal attention to many other matters. The welfare of his employees and their families engaged his attention to a marked degree, and insurance funds, schools and other evidences of his regard for his men have been frequently described by his biographers. It is characteristic of the man that even when labor troubles were in progress, and he was the victim of sympathetic strikes, the needy families of the men who were out found relief at his home. When Franklin B. Gowen was engaged in putting down the Molly Maguire lawlessness, Mr. Coxe was his hearty supporter and shared in his personal danger during those troublesome times.

Mr. Coxe was a Democrat in politics, but while he took an interest in public affairs, he was not personally active. He was elected a State Senator at one time, but had conscientious scruples against taking the oath of office, and did not occupy his seat until re-elected by an overwhelming majority.

He was an ardent believer in technical education and was a Trustee of Lehigh University from its beginning. In one of his last letters he wrote to a friend that he was living for two things—this university and the utilization of small coal. As a member of the Pennsylvania State

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Commission on "Waste of Anthracite Coal" and of the Commission in charge of Second Geological Survey of Pennsylvania he did much work, and aided the investigations of these important commissions with a lavish expenditure of time, money and privately acquired data. In recent years, his experiments looking towards the utilization of small coal have attracted much attention, and his apparatus for preparing and burning it have been described frequently.

Mr. Coxe was elected a Member of the American Society of Civil Engineers on February 7th, 1877. He was one of the founders of the American Institute of Mining Engineers, and was actively engaged in its management during twelve of the twenty-five years of his connection with it, which terminated only at his death. He was President of the Institute in 1878 and 1879, and held the same office from 1892 to 1894 in the American Society of Mechanical Engineers. He was also a member of a large number of other scientific and social clubs in this country and abroad.

Mr. Coxe died of pneumonia on May 13th, 1895, after a short sickness, and was buried in a little churchyard within a stone's throw of his offices in Drifton. His wife, an invalid for many years, two brothers and a sister, survive him.

HORACE LAFAYETTE EATON, M. Am. Soc. C. E.*

DIED NOVEMBER 23D, 1895.

Horace LaFayette Eaton was born in Boston, Mass., on September 6th, 1851. In 1869 he entered the office of the City Engineer of Boston as rodman, and subsequently became transitman, leveler and Assistant Engineer. He was employed at one time or another on all branches of work done under the direction of the City Engineer, both in the field and in the office, so that when he was elected in 1887 to the position of City Engineer of Somerville, he was well qualified for his duties.

Mr. Eaton had the reputation of being a very careful and cautious engineer and a man of strict integrity. All of his plans were worked out with a thorough regard for every detail. In the execution of all the public work constructed under his supervision he required honesty and faithfulness, and no work which was not up to the standard was accepted. False stories were ingeniously circulated by Mr. Eaton's enemies, and were used by certain cheap politicians who wished to control the office for their own purposes, and finally the City Council

* Memoir prepared by Desmond FitzGerald, M. Am. Soc. C. E.

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was induced to order an investigation into the affairs of the City Engineer's office.

The first hearing had been held, and his enemies had told their stories, all of which were printed in a sensational manner by the newspapers, before he had an opportunity to make any defence. Mr. Eaton, already overworked, was harassed and despondent. The false statements made in public so worked on his sensitive nature that, in a fit of temporary insanity, he took his own life.

The act came as a terrible shock to the community. The people of Somerville were sad and indignant, and requested the City Council to continue the investigation. It was carried on after his death in a most searching manner, the result being that Mr. Eaton was completely exonerated from every charge, and those making the false accusations were denounced for having brought them against an innocent man without any facts on which to base them.

Mr. Eaton was elected a Member of the American Society of Civil Engineers on February 1st, 1893. He left a wife and two children.

JOHN ROBERTS GILLISS, M. Am. Soc. C. E.*

DIED JULY 15TH, 1870.

John Roberts Gilliss was of Maryland ancestry, and was born in Washington, D. C., January 4th, 1842. He was the second of three sons of Capt. James M. Gilliss, U. S. N., who, as Lieut. Gilliss, conducted a U. S. naval astronomical expedition to the southern hemisphere in 1849–52, the results of which were published in an elaborate work as an executive document of the 33d Congress, and who, at time of his death in 1865, was in charge of the Naval Observatory at Washington.

The son, John R. Gilliss, had an unusually precocious intellect. His brain was large compared with his body, which was slight and delicate, although his appearance and bearing were in advance of his years. He received his education at a classical school in Georgetown, D. C., graduating at the early age of fifteen, when his teacher told his parents he could take him no farther in mathematics, and presented him with the only gold medal ever given in the school. Being very proficient in this study, he was, while yet a boy, a useful assistant to his father in astronomical computations.

In 1857 he went to Peru with W. W. Evans, M. Am. Soc. C. E., to take the place of Rodman on a railroad then being constructed under the direction of that noted engineer. In a short time, although but

* Memoir prepared by G. B. Nicholson, M. Am. Soc. C. E.

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sixteen years of age, he was promoted to the charge of a section of 16 miles under construction.

Returning to the United States, he entered the service of the Coast Survey as Aid, and afterwards was engaged during the greater part of the Civil War as a civilian assistant in the Engineer Department of the Army. From 1863 to July, 1865, he was principal Assistant Engineer to Lt.-Col. J. H. Simpson, Chief Engineer of the Department of the Ohio, being engaged all that time on the construction of fortifications in Kentucky. He had charge of the completion of those at Camp Nelson, and designed and built those at Frankfort and Louisville.

At the close of the Civil War, an engineer bureau was organized by the Secretary of the Interior to take charge of the Government interests in the Pacific railroads, and conduct the work of a number of wagon roads in the territories which Congress had placed in care of that department. Mr. Gilliss was made Principal Assistant, July, 1865, under the head of the bureau, Lt.-Col. Simpson, and rendered valuable aid in the preparation of the interesting report of his chief published in 1865. The discussion on the limiting effect of grades on railroads found in that report was conducted by Mr. Gilliss and was one of the earliest contributions to that subject published.

Desiring further experience in railroad engineering, he took a position in August, 1866, as Assistant to L. M. Clements, Resident Engineer of the Central Pacific Railroad, in the Sierra Nevadas, the work of Mr. Gilliss being principally on the tunnels of that section. A description of the tunnels of the Pacific railroads is found in a paper he read before the Society on January 5th, 1870.*

Having finished his work on the Central Pacific Railroad, he was employed as Engineer in charge of construction on the Union Pacific Railroad, between Wasatch and Castle Rock, on the completion of which he returned to New York, and was engaged on a pneumatic tunnel project in that city, which was never brought to a successful conclusion. He died suddenly of apoplexy of the brain at the home of his mother at West New Brighton, Staten Island, July 15th, 1870, aged 28 years.

Mr. Gilliss was elected a Member of the American Society of Civil Engineers on June 2d, 1869, and a Fellow on March 15th, 1870. He was versatile in his talents; it is not out of place to say, genius. A man of fine executive ability, he had the happy gift of commanding men without his subordinates realizing they were commanded; a charming companion, gentle and unobtrusive in manner, but nevertheless dignified; a constant reader of the best literature, and, while disclaiming that he was a mathematician, often reading works of pure mathematics with as much interest as others would fiction; and notwithstanding his mathematical bent, he had a large artistic talent.

His early death prevented his name from acquiring the renown as an engineer which was so largely promised.

* See *Transactions*, Vol. I, p. 163.

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WILLIAM HARRISON GRANT, M. Am. Soc. C. E.*

DIED OCTOBER 12TH, 1896.

William Harrison Grant died at his home in Sing Sing, N. Y., Saturday, October 12th, 1896, aged eighty-one years and five months, having been born May 15th, 1815. He was descended from William Grant, a revolutionary soldier who was pensioned for his services in both the army and the navy.

When nineteen years old, Mr. Grant entered the New Paltz Academy, where he studied mathematics and surveying. He began practice in country surveying and on the New York and Erie Railroad, after which he went to Ithaca Academy to continue his mathematical studies and complete his education. Soon after leaving the academy he was appointed Assistant Engineer on the enlargement of the Erie Canal under William J. McAlpine, where he remained some eight or nine years. During intervals in his canal service, he spent two winters in the Legislature at Albany as Deputy Clerk, and also acted as assistant to Adjutant General Niven. Later he was employed as Assistant Engineer on the Hudson River Railroad survey under John B. Jervis, and before that road was completed, he was appointed Chief Engineer of the Cleveland, Zanesville and Cincinnati Railroad, of which he constructed 60 miles. He was then called to Washington, D. C., and took charge of a railroad survey from Georgetown to Hagerstown, Md. This road was not built at the time, but the surveyed route was afterwards occupied by the Baltimore and Ohio Railroad.

Returning to New York, he was appointed Superintending Engineer of Central Park, and was connected with the work until its completion. In later years, he looked back upon this portion of his professional career with the greatest pride. Following his connection with Central Park, he was appointed Superintending Engineer of the Department of Public Works of New York City, his charge embracing the annexed district, across the Harlem River, now the 23d and 24th Wards of the city. He afterwards became Constructing Engineer of the Department of Public Parks. In 1876 Mr. Grant formed a partnership with Donald G. Mitchell, as landscape architects, but in a year or two he accepted an appointment from the general government on the improvement of rivers and harbors in Maryland and Virginia.

Mr. Grant had a large and varied experience in engineering work, and in addition to the engagements already noted, there were terms of service upon the Eastern Division of the Ontario and Western Railroad, and the northern half of the West Shore Railway. He also furnished the plans and constructed in part the Yonkers system of water

* Memoir prepared by Alfred P. Boller, M. Am. Soc. C. E.

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works. His last public place was as Superintendent of the new United States Naval Observatory, Mr. Richard M. Hunt, architect. On the near completion of the several buildings under his charge, he resigned his position, retiring in 1893 from the active practice of his profession, and passing the remainder of his life at Sing Sing, N. Y.

Mr. Grant was one of the early members of the American Society of Civil Engineers, which he joined on July 2d, 1873. In early life he married Miss Mary Locke, of Sing Sing, daughter of James Locke, and sister of Rev. Dr. Clinton Locke, of Grace Episcopal Church, Chicago. She survives him, together with three children: Dr. Frank S. Grant, of New York City; Mrs. James E. Childs, wife of the General Manager of the Ontario and Western Railway, and Miss Teresa Grant. Mr. Grant was one of the very last of the old-school engineers, with whom engineering was wholly a development from subordinate field positions. In their youth the technical school was unknown, and there was no engineering literature to draw upon. They were bold and resourceful men, who accomplished much in their day and generation, and paved the way well for the great later-day triumphs of the profession.

The probity of character that attached to these men was highly marked in Mr. Grant, whose uncompromising integrity was one of his strongest characteristics, although coupled to a most kindly disposition.

ROBERT G. HATFIELD, M. Am. Soc. C. E.*

DIED FEBRUARY 15TH, 1879.

Robert G. Hatfield, born at Elizabeth, N. J., in 1815, began his career as a carpenter and builder. He was of a studious nature, and soon fitted himself to be an architect. He designed a number of structures, chiefly warehouses and shops, but achieved his reputation as an architectural engineer. His designs for complete buildings have been criticized as not highly artistic in character, but sometimes very ingeniously planned and always well executed. In the technicalities of his profession, however, he was regarded as a recognized authority, and as his spare time for some years was spent in making experiments on the strength of building materials with a testing machine of his own invention, he acquired a large amount of data on this subject. He was a painstaking investigator of matters relating to the computation of stresses, and among his books is one on "The Theory of Transverse Strains and its Application in the Construction of Buildings." In this book are a large number of original tables based on the results of his

* Memoir prepared from papers on file at the House of the Society.

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investigations. An account of some of his experiments with stones of various classes is given in a paper presented by him to this Society at its fourth annual convention and printed in the second volume of *Transactions* under the title of "Experimental Tests of Building Stones." Among his earlier books was one entitled "The American House Carpenter," which had a considerable sale fifty years ago.

As a designer of structural work, Mr. Hatfield was frequently consulted by other architects, his best known work probably being the arched roof of the Grand Central Station at New York City. He was also frequently retained as a referee in disputed subjects, and was employed by Andrew H. Green, when Comptroller of New York, to estimate the value of the work actually done on the County Court House in that city by the contractors in the Tweed ring.

Mr. Hatfield was elected a Member of the American Society of Civil Engineers on December 4th, 1867. He was one of the earliest members of the American Institute of Architects and an earnest worker in advancing its growth and influence. He gave up much time to its affairs, acting as Treasurer for many years and serving as President of the New York chapter for some time. Among his papers presented to that body were two advocating improvements in construction tending to diminish danger from fires, a subject to which he paid much attention.

Mr. Hatfield died in Brooklyn on February 15th, 1879.

NORMAN JAMES NICHOLS, M. Am. Soc. C. E.*

DIED APRIL 8TH, 1896.

Norman James Nichols was born in Essex, Vt., December 3d, 1835. He graduated from the Chittenden County Institute at that place in 1858, having taken a course in engineering, and intended to go West immediately to engage in railroad work. These plans had to be abandoned, however, and until 1862, he was engaged as carpenter and building contractor in Essex and neighboring towns. In August, 1862, he enlisted with the Second Vermont infantry and served in many severe engagements, being wounded at the Wilderness. In May, 1865, he was honorably discharged, and began work again as a contractor for buildings and highway bridges in Essex. He followed this business for about eight years, with occasional intermissions during which he was engaged as draftsman or superintendent in manufacturing establishments.

In the winter of 1873–74 Mr. Nichols moved to Worcester, Mass., and became associated with the firm of Buttrick & Wheeler, with

* Memoir prepared from papers on file at the House of the Society.

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whom he remained for three years. This firm had a general engineering practice at that time, and was engaged in laying out the sewerage, sewage disposal and water systems for the large State Lunatic Asylum in Worcester; surveys for extensions of the Worcester and Shrewsbury Railroad; the location of the Connecticut Valley Railroad, a narrow-gauge line from New Haven, Conn., to Turners Falls, Mass., and general city surveying. In 1878, Mr. Nichols was in charge of the construction of a water-power dam at Cherry Valley, Mass., to replace a structure destroyed by the Lynde Brook Reservoir disaster, which forms the subject of a report in *Transactions*, Volume V, page 244.

In February, 1879, Mr. Nichols went to the Republic of Colombia to take part under Daniel M. Wheeler in the surveys and construction of the Magdalena Railroad, about 30 miles long, connecting the navigable reaches of the Lower and Upper Magdalena River around the falls at Honda. In about a year the company engaged in the work failed, and he returned to the United States. In July, 1881, he was engaged by F. J. Cisneros, M. Am. Soc. C. E., to take the place of an engineer retiring from the staff working on the Antioquia Railroad, a 125-mile line from Puerto Berrio to Medellin, and was soon appointed Chief Engineer of that undertaking. From April, 1882, to May, 1886, he was Chief Engineer of the La Dorada Railroad, and afterward, up to the time of his death on April 8th, 1896, was engaged in general engineering and mining work in the vicinity of Honda, Republic of Colombia.

Mr. Nichols was elected a Member of the American Society of Civil Engineers on December 5th, 1888.

ALBERT FRANKLIN NOYES, M. Am. Soc. C. E.*

DIED OCTOBER 12TH, 1896.

Albert F. Noyes, son of George H. Noyes, was born in South Boston in 1850. The family removing to Melrose, he was educated in the public schools at that place, and prepared for his special career at the Lawrence Scientific School of Harvard University, Cambridge, Mass. From July, 1871, to November, 1873, he was an Assistant in the office of Ernest W. Bowditch, Jun. Am. Soc. C. E., and Mr. Charles H. Bateman, engaged chiefly on topographical surveying and landscape architecture. Subsequently he was Principal Assistant to Mr. Frederic Schoff, City Engineer of Newton, Mass., and from March, 1875, to February, 1876, was Acting City Engineer of that place.

* Memoir prepared by H. D. Woods, M. Am. Soc. C. E.

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He was appointed City Engineer of Newton in February, 1876, and filled that position for seventeen years. During this long term of office the development of the city constantly increased the demands made upon the office. Besides having charge of all the city surveying, making all plans for public works, the City Engineer had the laying out of all works on the highways and had to keep the accounts of all highway expenses, and was engineer of the Water Board. In 1882 he was appointed Plumbing Inspector for the Board of Health, which position he filled for several years, remodeling the plumbing regulations. In 1882–83 he was called upon by the Public Property Committee to make a sanitary survey of all the school houses, and designed and executed plans for heating and ventilating several of the old buildings. He mapped out a proposed plan for a 376-acre park in the central part of the city. He inaugurated a system of highway accounts, whereby the actual cost of each class of work was kept distinct, thus enabling him to make accurate estimates of any work proposed to be done by the Department. In 1884 he made a sanitary inspection of the city with reference to sewer requirements, and in 1887, with members of the city government, visited the separate system of sewers as constructed in various towns and cities in New Jersey and Pennsylvania, also Pullman, Ill. In 1890 he made a comprehensive report on a separate system for the city of Newton, which included 130 miles, estimated to cost $1 750 000, or an average of $2 56 per foot. The system was adopted and construction commenced the following year, over 40 miles being built under his administration. In 1889 in connection with Alphonse Fteley, M. Am. Soc. C. E., as Consulting Engineer, he made a report on the additional water supply for the city of Newton, and also on a high-service system. In 1892 the additional supply work was carried out by building a covered filtering conduit and the first covered masonry reservoir in this section of the country. In 1892 in connection with Mr. Edward Buss, he made an exhaustive report on a system of surface drainage for the whole city, which included the improvement of the various brooks, forming a series of parkways through the city. Nearly one mile of Cheese Cake Brook was straightened and deepened and laid out with drives on either side under this proposed improvement. Early in 1893 in connection with Charles A. Allen and George S. Rice, Members Am. Soc. C. E., he reported on a plan for abolishing the various crossings on the Boston and Albany Railroad through the city. Plans for several miles of boulevard 120 ft. wide were prepared the same year, but, owing to legal questions, actual work was not begun during his administration.

On July 24th, 1893, Mr. Noyes resigned his position as City Engineer of Newton to accept the position of Assistant Chief Engineer of the Massachusetts State Board of Health, to make a special study of the ground-water supplies of the State. During 1893, 1894 and 1895

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he examined and reported on the water supply of forty or fifty places in Massachusetts. He also investigated the availability of the Ipswich and Shawsheen Rivers and the water-shed of the Charles River for a portion of the metropolitan supply. From 1886 to 1889 he investigated and reported on systems of ground-water supply for the cities of Malden and Brockton and the towns of Reading and Medford, and also on the distributing system for the Milford Water Company.

In 1894 he was appointed by the Governor a member of the Metropolitan Sewer Commission, to succeed Mr. Harvey N. Colligan. This was an especially appropriate appointment, as from its start Mr. Noyes had been conversant with the work and plans of the metropolitan sewer construction, and the requirements of the outlying districts, the city of Newton being one of the first to connect with the system. In 1895 he was elected to represent his ward in the Board of Aldermen of the city of Newton, and was appointed on the Highway Committee and Chairman of the Sewer Committee.

He closed his connection with the State Board of Health, and in February, 1895, formed a partnership with Allen Hazen, Assoc. M. Am. Soc. C. E., under the firm name of Noyes & Hazen, making a specialty of sewer and water-works investigations and construction. In the line of water supply this firm made investigations, surveys or reports for Austin, Tex.; Far Rockaway, L. I.; Menominee, Mich.; Columbus, Cleveland and Painesville, O.; Harrisburg and Du Bois, Pa.; Jersey City, N. J.; Ashland, Wis.; Indianapolis and Princeton, Ind.; Greenfield, Munson, Lynn and Adams, Mass.; and for a high service supply for Lawrence, Mass., and Hartford, N. Y. In the line of sewerage it was connected with the design or construction of sewer systems in Altoona, Pa.; Plainfield, N. J.; Vassar College, Poughkeepsie, N. Y.; Spencer, Quincy and Melrose, Mass.

Mr. Noyes showed remarkable ability and excellent judgment. He was a broad-minded public official, and a very careful, conscientious worker, adhering strictly to the course which he considered best for the city. He was a constant student, and never undertook any work without examining the subject from all sides, and ascertaining how to obtain the best results with the least expenditure of public funds. As an example of this, when the work was commenced on the Newton sewer construction, and the various pipe dealers had made a combination and fixed prices, he visited, with the Chairman of the Sewer Committee, the principal pipe yards in the Akron and Ohio River districts, St. Louis and the Potomac Valley, to ascertain the ability of the yards to furnish the material required, and to get information which would allow him to better judge the quality of the pipe required. He was able in this way to make contracts with firms outside the combination, and to get some of the best pipe that was ever used in an eastern market at a considerable saving to the city. What-

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ever matters were brought to his attention were thoroughly investigated, and whatever knowledge was required outside of his own experience was carefully looked up, and the advice of the best authorities sought before settling on a definite policy.

He was generous and kind, had a pleasant greeting for everyone, and would listen patiently and attentively to all grievances brought to his attention, endeavoring to give everyone justice, and to repair all apparent wrongs which might be caused to private individuals through the carrying out of public works. By his death the city of Newton lost a valuable public-spirited official and citizen, who was ever mindful of the welfare and development of the city. The State lost a valuable adviser, and the engineering profession one of its most prominent members, he being considered one of the foremost engineers in his specialty. He was regarded as an expert in matters of ground-water supply and sewerage, his advice being sought from various parts of the United States.

Mr. Noyes was elected a member of the American Society of Civil Engineers on December 3d, 1884. He was a Member and Past-President of the Boston Society of Civil Engineers, and at the time of his death was Vice-President of the Massachusetts Highway Association. He was President of the New England Water-Works Association for the years 1890 and 1891. He leaves a widow, one son and two daughters.

HENRY WARD BEECHER PHINNEY, M. Am. Soc. C. E.*

DIED NOVEMBER 22D, 1888.

Henry Ward Beecher Phinney was born in Jay, Me., January 28th, 1854.

In May, 1875, he entered the office of Shedd & Sawyer, Civil Engineers, in Boston, Mass., as student and apprentice, and was employed on the construction of water-works at Newton, Mass., and in charge of pile-driving on the main sewer at Brookline, Mass.

From 1878 to 1882 he was employed under Howard A. Carson, M. Am. Soc. C. E., on the construction of the improved sewer system of Boston, Mass., as Assistant Engineer and Foreman, having charge of many important structures. In March, 1882, he took charge of the construction of the receiving well for the Indianapolis, Ind., Water Company, 12 ft. in diameter and 30 ft. deep, through saturated gravel on the bank of White River, under the direction of J. J. R. Croes, M. Am. Soc. C. E. On the completion of this work he was employed

* Memoir prepared by J. J. R. Croes, M. Am. Soc. C. E.

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by Mr. Croes as Resident Engineer of the construction of the water-works at Princeton, N. J., and afterwards, from 1883 to 1886, as Resident Engineer of the Suburban Rapid Transit Company, having charge of the surveys and the construction of the Harlem River Bridge, and the brick and iron viaducts in Morrisania, New York City.

Pulmonary disease compelled him to resign this position on December 1st, 1886, and he went to Pasadena, Cal., where he died on November 22d, 1888.

Mr. Phinney was accurate and painstaking in all his work, and was an excellent organizer and superintendent of labor, a good draftsman and computer, and a rigid and at the same time judicious supervisor of the construction of works.

He became a Member of the Boston Society of Civil Engineers in October, 1879, and of the American Society of Civil Engineers on January 7th, 1885.

THOMAS PROSSER, M. Am. Soc. C. E.*

DIED SEPTEMBER 15TH, 1870.

Thomas Prosser was educated in England as a civil and mechanical engineer and architect, and practiced his profession for some time previous to his removal to the United States, which took place about 35 years ago. In this country he has been better known as a mechanical engineer and inventor, and was for several years engaged in the manufacture of boiler tubes.

An acquaintance which he formed with the proprietor of Krupp's celebrated steel works at Essen, Prussia, while visiting the London Exhibition of 1851, resulted in the appointment of Mr. Prosser to the American agency of those works, which he held until his decease. For several years past Mr. Prosser was the only agent remaining in position from among those who had received appointment by the personal selection of Mr. Krupp.

In the autumn of his life Mr. Prosser became a member of the American Society of Civil Engineers and Architects soon after its reorganization in 1867, his name being the tenth in order on the roll of new members.

He brought into our circle a thoughtful, inventive and methodical mind, well stored with useful knowledge, and, disciplined by long experience and culture, soon took a prominent place, and was during his entire membership thoroughly identified with the growth and welfare

* Memoir prepared by Messrs. Jacob M. Clark, John F. Ward and A. W. Craven, in 1870, but never published.

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of the Society. His name is connected with many of the important business committees which have been formed, and for nearly a year preceding his decease he was a member of the Board of Direction. His duties were always assumed without ostentation, and most faithfully and punctually discharged. The minutes show that no living member has been for the same period a more constant attendant on our meetings, and his colleagues in committees and on the Board remember that his place was never vacant unless when he was detained by infirmity, and that he always came thoroughly prepared for business. His sincere convictions and thorough discipline made him a tenacious, though always courteous and manly, opponent, and as an ally, staunch and reliable.

The published *Transactions* of the Society are enriched by an interesting article from his pen, illustrating a highly ingenious method and apparatus of his invention for the conservation of heat in distillation.

Few members have bestowed a more liberal share of honest and earnest effort upon the business of the Society than Mr. Prosser.

With less of evil to "live after him" than can be said of most men, his spotless record may well be our just pride, so that the good may not be "buried with his bones."

This Committee recommends the following preamble and resolutions:

Whereas, The American Society of Civil Engineers and Architects, by decree of that Divine Power whose mandates all must obey at the appointed time, has been deprived of a valued and honored member and officer, skilful and distinguished in his calling, able and intrepid in counsel, a genial and sympathizing friend, worthy in all the relations of life; therefore

Resolved, That we tender to the surviving family and friends of the late Thomas Prosser our cordial sympathy and condolence.

WILLARD SMITH POPE, M. Am. Soc. C. E.*

DIED OCTOBER 10TH, 1895.

Willard Smith Pope was born on January 16th, 1832, in the then village of Rome, N. Y. He was the son of Dr. G. W. Pope, a physician and a man of standing in the community. His early life was spent in Rome, where he attended school and prepared for college. He entered Hamilton College, Clinton, N. Y., in 1847, and graduated

* Memoir prepared by George S. Morison, Past-President Am. Soc. C. E.

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in 1851, at the early age of nineteen. After graduation he went to Buffalo, where he studied law, and was admitted to the bar after a single year of hard work. The early labor by which he had accomplished this result at a period when boys are often still at school now told upon him, and his health gave way. He left Buffalo, returned to his father's home in Rome, and spent several months in the open air, under the general care of his father.

Feeling that his health required an outdoor life, he obtained a position in the engineer corps of the European and North American Railway, and spent the fall of 1852 and the following winter in the woods of New Brunswick and Maine. This was the beginning of his work as an engineer, and he decided to abandon the law and make engineering his profession. He next spent the larger part of a year in the Astor Library in New York City, laying a theoretical groundwork for his new profession. In 1853 he went to the West, and worked for the Illinois Central Railroad on the location of its line in Southern Illinois. From there he went, in 1854, to the Galena and Chicago Union Railroad, which became the Chicago and Northwestern Railway in 1864 by a consolidation with other lines. He remained there until the latter part of 1864, and during the last four or five years he held the position of Chief Engineer.

It was while there that he was called on to do his first important work in bridge building. The bridge across the Mississippi River at Clinton was the second bridge across that river, the only earlier one being the old Rock Island bridge, the piers of which were founded on rock in shallow water. No such rock existed at Clinton, and the river was narrow and exceptionally deep. Mr. Pope used pile foundations, which have since become the standard practice on the upper Mississippi for all the piers except the pivot piers, and founded the pivot pier on a timber crib 400 ft. long, which not only carried the pier, but formed the draw protection. The same pier is still in good condition on the same foundation.

The draw of the Clinton Bridge was the first iron draw of importance built in the West; it consisted of two spans of Bollman trusses hung by hog chains from a central tower; it was built by contract by the Detroit Bridge and Iron Works. This contract shaped the remainder of Mr. Pope's life. He resigned his position as Chief Engineer of the Galena and Chicago Union Railroad, associated himself with the Detroit Bridge and Iron Works, and was elected a Director of that company on February 9th, 1866. He opened an office in Chicago and remained there a year as the representative of the bridge works; he then went to Detroit and took direct charge of the engineering department. On May 7th, 1869, he was elected President of the company, and held this office until his death.

Mr. Pope's connection with this company, covering a period of

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nearly 30 years, so completely identified the man and the corporation that no life of Mr. Pope is complete which does not include a history of the bridge works. The Detroit Bridge and Iron Works was incorporated in the year 1863, and succeeded to the business of the bridge-building firm of Charles Kellogg & Co., this firm consisting of Mr. Charles Kellogg, now deceased, and Mr. William C. Colburn, who is still treasurer of the company. As early as 1861 this firm had constructed iron bridges on the Illinois Central Railroad and on the Galena and Chicago Union Railroad; they were of the Bollman pattern, and may be considered the pioneer iron bridges built in the West.

From February, 1866, to the time of his death, with occasional short vacations, Mr. Pope gave the most thorough personal supervision to all the work constructed by this company. In 1866 the Detroit Bridge and Iron Works took the contract for the superstructures of the bridges at Burlington and at Quincy, these being the first two all-iron bridges built across the Mississippi River; they also contained the first draw bridges of what may be called modern dimensions, and Mr. Pope's ability as an engineer was strikingly illustrated in the special features of these draws; their turn-tables were far in advance of others built up to that time, and they were equipped with a system of lifting cams at the ends which are still as good as anything in use; both these bridges were opened for traffic in 1868.

In 1869 and 1870 the Detroit Bridge and Iron Works built the bridge across the Mississippi River at Hannibal, taking the entire contract for both substructure and superstructure, the first time this had been done on any great western bridge. Subsequently the same company took the contract for the bridge across the Missouri River at St. Joseph, which was opened in 1873, building the entire substructure and superstructure. Col. Eddy D. Mason, M. Am. Soc. C. E., was the chief engineer of both of these bridges, and they are both monuments of the skill of the engineer proper and the engineering contractor. Although no complete bridges of equal magnitude were subsequently built by the Detroit Bridge and Iron Works, the company has continued steadily in the bridge business, confining itself principally to superstructure. Its shops at Detroit have always ranked among the best class of bridge shops, and engineers have always felt that Mr. Pope intended to furnish the best work his shops could produce.

Among the last important works constructed by this company may be mentioned the Ferris Wheel for the World's Columbian Exposition and the steel gates for the new lock on the St. Mary's Falls Canal.

Socially Mr. Pope was one of the most delightful of men. Always chary about asserting himself, he had to be drawn out; but the drawing process disclosed his possession of a fund of the quaintest

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humor, which became on occasions exceedingly bright and sparkling. He was especially happy in a certain affectation of cynicism which was oddly at variance with his real habit of thought, and used it very effectively in the puncturing of shams, which was somewhat of a passion with him. He was always, however, tenderly careful of the feelings of others, and resented with all the force of his really strong nature the reckless habit into which so many have fallen in recent years of trifling in speech and otherwise with the reputation of others. He was not given as a rule to impassioned utterance; but those who knew him best will recall more than one occasion when he indulged in it in denunciation of criticisms upon individuals which he deemed unwarranted, inexcusable and slanderous.

Mr. Pope loved the quiet of his home and the companionship of those dearest to him. In that circle he was both loved for his earnest affectionate guidance as husband and father, and respected and looked up to for his wisdom, his learning, his keen sense of justice, and his generous appreciation of all with whom he was closely associated. In the life of his own home the brightest and sweetest parts of his character were apparent. His life was entirely free from those unfortunate deficiencies of character and want of self control that often wear out the cheer and affection which are the necessary conditions of all true home life. His presence brought protection and happiness to his family and filled his home with cheerfulness and light.

He loved and was well acquainted with the best English literature, and this, with an unusual memory and facility of quotation, made him a most agreeable and instructive companion. During his residence in Detroit he was much sought after as a bright and witty speaker at public dinners.

Mr. Pope was married three times: In 1856 to Miss Harriet L. Bissell, daughter of Dr. Emory Bissell of Norwalk, Conn.; she died in the following year. In 1861 he married Miss Julia Bissell, a sister of his first wife; she died in 1872. In 1882 he married Mrs. Martha E. Patterson, widow of Philo M. Patterson of Detroit and daughter of W. B. A. Bissell, Bishop of Vermont, who survives him. He leaves three daughters and one son, all of whom, with his widow, reside in Detroit. His son, Willard Pope, has followed his father's profession, and is now an engineer with the Detroit Bridge and Iron Works.

Mr. Pope became a member of the American Society of Civil Engineers August 7th, 1872; he was elected a Director in January, 1893, and had nearly completed his term of office at the time of his death.

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JAMES CLARENCE POST, M. Am. Soc. C. E.*
Major, Corps of Engineers, U. S. Army.

DIED JANUARY 6TH, 1896.

James Clarence Post was born at Newburgh, N. Y., July 30th, 1844. In 1861 he entered the Military Academy at West Point, and on graduating four years later was appointed Second Lieutenant in the Artillery. He was soon promoted to First Lieutenant and transferred to the Corps of Engineers, with which he was afterward connected until his death. After acting for a short time as Assistant Engineer in the improvement of government works in New England, he was appointed Assistant Professor of Mathematics at the Military Academy at West Point, where he remained for four years. The next three years were spent with the engineer battalion at Willet's Point, N. Y. From June, 1874, to November, 1882, he was Assistant Engineer under Lieutenant-Colonel Gillmore, and in temporary charge of his works to April, 1883. The next four years were spent in charge of river improvements in several states.

In May, 1887, Major Post was transferred to Washington as assistant to the Chief of Engineers, and in July, 1889, he became Military Attaché to the United States Legation at London. In July, 1891, he was a delegate to the Geographical Congress at Berne, Switzerland, and in 1893 attended the International Maritime Congress held in London.

From February, 1894, to December, 1895, Major Post was in charge of important works in the Northwest. The large jetty at the mouth of the Columbia River was completed under his supervision, and his work in 1894, during a great flood in that river, when the Cascades Canal was in danger of serious injury, was highly commended by an examining board composed of General William P. Craighill, Past-President Am. Soc. C. E.; Colonel George H. Mendell, M. Am. Soc. C. E., and Major Marshall. During this time he was also in charge of the construction and equipment of a boat railway from the foot of the Dallas Rapids to the head of Celilo Falls, and of many river and harbor works. In addition to these duties he was engineer of the Thirteenth Lighthouse District, served as a member of boards on bridge construction and river and harbor improvements, and was in supervisory charge of the construction of bridges across various streams.

In December, 1896, he was relieved from duty at Portland and detailed as the successor of the late Orlando M. Poe, M. Am. Soc. C. E., at Detroit, and was about to assume his new duties, when he died.

Major Post was promoted to Captain in October, 1871, and to Major in September, 1886. He was married at London in 1892, and leaves a widow and infant son.

He became a Member of the American Society of Civil Engineers on February 6th, 1878.

* Memoir prepared in the office of the American Society of Civil Engineers.

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HENRY FREDERICK RUDLOFF, M. Am. Soc. C. E.*

DIED JUNE 1ST, 1895.

Henry Frederick Rudloff died at Caracas, Venezuela, June 1st, 1895. At the time of his death he was Chief Engineer of the Puerto Cabello and Araure Railway, and was also in the service of the Venezuelan government in charge of various minor engineering works.

He was born in Prussia, September 12th, 1846; graduated from the Gewerbe Institute in Berlin in 1866, and from that time until 1878 was engaged upon various public works in different parts of Europe, his father being a prominent constructor. He came to America in 1878, and during the next three years had several engineering engagements, among them office duties with Clarke, Reeves & Company, of Phœnixville, Pa.; with Gen. McClellan on a projected underground railway, and with the West Shore Railroad. In 1881 he entered the service of the government of Venezuela, and became Division Engineer on the La Guayra and Caracas Railroad, then in progress of construction, having charge of tunnels and bridges upon that road. In 1883 he was appointed Government Engineer at Caracas and designed a number of works, including a notable stone arch and an iron viaduct in Caracas. In 1884 he became the engineer of an American company, and under his direction the works of improvement of the harbor at Puerto Cabello were carried out. He was also the engineer of the Caracas and Antimano Railroad, and continued to the time of his death in government service, in addition to his engagement with the American company.

Mr. Rudloff was a man of exceptional ability in the direction of public works in Spanish-American countries. He had command of several languages, and was able to apply the results of varied experience to the special requirements of construction in those countries. He was entrusted, both by the Venezuelan government and by the representatives of foreign capital in that country, with the design and construction of engineering work of considerable magnitude, and always with satisfactory results. His death was sudden and unexpected, coming at a time of promise of much more extended possibilities. He leaves a widow and one son, who reside at Caracas, Venezuela.

He was elected a Member of the American Society of Civil Engineers January 6th, 1886.

* Memoir prepared by John Bogart, M. Am. Soc. C. E.

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ISAAC MUNROE ST. JOHN, M. Am. Soc. C. E.*

DIED APRIL 7TH, 1880.

General Isaac Munroe St. John, eldest child of Isaac R. and Abby R. (Munroe) St. John, was born November 19th, 1827, at Augusta, Ga., where his father was then engaged in business. He graduated from Yale College with the degree of B. A. in the class of 1845, and received his degree of M. A. from that college in 1848.

On graduating he began the study of law in New York, but removed to Baltimore in 1847, where he was employed as Assistant Editor of the *Patriot*. He subsequently chose civil engineering as a profession, and was connected with the engineering corps of the Baltimore and Ohio Railroad until 1855. In that year he removed to Georgia, and was in charge of divisions of the Blue Ridge Railroad for five years.

In February, 1861, he entered the Confederate service as a private in the Fort Hill Guards of the South Carolina State troops. Two months later he was transferred to engineer duty, and rose rapidly to the position of Chief Engineer of the Army of the Peninsula. In May, 1862, he was made Major and Chief of the Mining and Nitre Bureau Corps. He was promoted through the various grades to the rank of Brigadier-General, and in 1865 to the position of Commissary General of the Confederacy.

After the war he resumed his profession, and from 1866 to 1869 was Chief Engineer of the Louisville, Cincinnati and Lexington Railroad. In 1870 and 1871 he was City Engineer of Louisville, Ky. From 1871 until his death he was Consulting Engineer of the Chesapeake and Ohio Railroad.

He was elected a member of the American Society of Civil Engineers on July 14th, 1871.

During the war he married a daughter of Colonel J. L. Carrington, of Richmond, Va. He died suddenly at his residence at White Sulphur Springs, W. Va., April 7th, 1880, aged fifty-two years.

* Memoir prepared by Edwin A. Hill, M. Am. Soc. C. E.

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HOWARD SCHUYLER, M. Am. Soc. C. E.*

DIED DECEMBER 3D, 1883.

Howard Schuyler was born at Ithaca, N. Y., December 11th, 1844. He received a thorough academic education at the Ithaca Academy, and at the age of fourteen was preparing to enter college when the family removed to Kansas in the spring of 1859. His studies were interrupted by this removal to the frontier, although his faculty for intense concentration enabled him to cover a wide range of scientific reading under adverse circumstances, and he absorbed knowledge with unquenchable thirst and avidity. Two years of severe physical toil and hardship preceded the outbreak of the War of the Rebellion, and in May, 1861, he enlisted as a private soldier in the 2d Kansas Regiment. He was then little more than sixteen years old, but so mature of mind and physique that he easily passed muster as a man of legal age. Under General Lyon he was in the battles of Forsyth, Dug Springs and Wilson's Creek, after which his regiment, having served the six months of its enlistment, was disbanded, and he re-enlisted in the 11th Kansas Infantry, which was attached to the army of General Blunt, then in Arkansas, and participated in the battle of Cane Hill in November, 1862, and of Prairie Grove the month after. In January, 1863, he was commissioned First Lieutenant for bravery in the field, but declined the commission because of his youth and inexperience, and in the June following he was appointed Lieutenant of Artillery for conspicuous bravery, which he also declined. Three months later he accepted the position of Captain in the 11th United States Colored Troops, and, after bringing his company to a high degree of discipline, he was transferred to 4th Arkansas Cavalry as First Lieutenant, and afterwards was promoted to the rank of Captain, which he held until the close of the war. He was then offered a commission in the regular army, but declined to accept it, preferring to practice the profession which he always had in view, and for which he had been fitting himself. In all of his army service he passed through numerous pitched battles, where the fighting was almost hand to hand, but escaped without a scratch, although his clothes were riddled by bullets.

Shortly after leaving the army he entered the service of the Kansas Pacific Railway as rodman, while the road was being constructed between Lawrence and Topeka, Kan. He won promotion rapidly, and successively became leveler, transitman, and Locating Engineer in charge of a party. In 1867 he was put in charge of one of five parties

*Memoir prepared by James D. Schuyler, M. Am. Soc. C. E.

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selected to survey the route of the thirty-fifth parallel road across the continent, under General William J. Palmer, and carried his line through to the Pacific Coast, completing the survey in the spring of 1869. It was an expedition requiring nerve, bravery and physical endurance in every member of the party, as they were harassed on all sides by Indians, and were subject to the hardships of thirst and starvation. On the completion of this survey he resumed charge the location of the main line of the Kansas Pacific, from Fort Wallace to Denver, about 200 miles, and completed its construction to that city in 1870. It was during this construction that he suffered the remarkable experience of having a locomotive pass over his foot, crushing all the bones of the toes, and yet recovered the complete use of the member. After his recovery he was promoted to Assistant Manager and Paymaster, which position he filled until the completion of the road to Denver.

During all these years of work on the location and construction of the Kansas Pacific Railway, the surveying parties were continually harassed by hostile Indians, and were frequently obliged to fight for their lives. His army life and acquaintance with warfare was admirable preparation for this experience, and on one memorable occasion, June 19th, 1869, Mr. Schuyler, while leading the advance of a preliminary survey near Sheridan, Kan., and while engaged in picking out a line for the party a few miles in the rear, was surprised and surrounded by a band of 100 or more hostile Cheyennes, but by coolness and presence of mind, he cut his way through the lines and escaped without a scratch, killing four Indians in the engagement. Five bullets were lodged in his horse; his field-glasses and one spur were cut off by bullets, and his clothes were well riddled; even the handle of his carbine was pierced and nearly torn from his grasp by a rifle ball. His plucky defence of himself saved the lives of the rest of the party, and they all succeeded in retreating to the military post fifteen miles distant, fighting their way through the swarming hordes of savages, whose well-laid plan for picking off the party one at a time was baffled by the coolness of their intrepid leader. The only member of the party wounded in this thrilling adventure was the writer.

In 1871 he became one of three organizers of the Denver and Rio Grande Railway, and was appointed its Secretary and Treasurer. In its service he visited Europe with General Palmer to examine the narrow-gauge railways of Wales and to interest foreign capitalists in its construction, in which they were highly successful. He afterwards filled the position of Chief Engineer and Assistant General Manager until May, 1873, when he resigned to accept the position of Chief Engineer of the North Pacific Coast Railroad, in California, and constructed the road from Sausalito, opposite San Francisco, to the Russian River, its northern terminus for many years subsequently.

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In June, 1880, Mr. Schuyler was appointed Chief Engineer of the Mexican Central Railway, with headquarters at the city of Mexico, and located and constructed the road as far as the city of Leon. The strain which this work imposed upon his physical resources can only be appreciated by those who have undertaken the task of railroad construction in a foreign country, dependent upon a class of labor the most ignorant, inert and impassive on earth, with no conception of energetic completion of a task undertaken, but with innumerable feast days to be celebrated, during which all work must be suspended. Overwork and exposure in a malarious climate undermined his health and obliged him to retire and take a trip abroad in the spring of 1883. He was accompanied by his family and visited various health resorts and consulted eminent specialists, but finally succumbed to a complication of diseases at Davos, Switzerland, December 3d, 1883, leaving a wife and one son, the latter born in the city of Mexico.

Mr. Schuyler was a man distinguished among all who knew him as one of rare personal charm and magnetism, making hosts of friends wherever he went or with whomsoever he was thrown in contact. This enviable gift, combined with a natural faculty he possessed as a leader of men, and executive ability and capacity for systematic organization of an unusual order, constituted the secret of his success and gave promise for a brilliant future could he have rounded out the full measure of life. A keen sense of honor and dignity influenced all his actions and compelled his recognition as a gentleman in every sense that word implies.

THOMAS JENNINGS SEELY, M. Am. Soc. C. E.*

DIED OCTOBER 2D, 1883.

Thomas Jennings Seely was born at Chester, Orange County, N. Y., August 16th, 1848. He was graduated from the University of Michigan in June, 1869, in both civil and mining engineering, and in a few months began active work in the engineering corps of the La Clede and Fort Scott Railroad Company. In 1871–72 he filled an engineering position on the Decatur and State Line Railroad, and in 1876 was Chief Engineer of the Chicago and Millington Railroad.

In April, 1878, Mr. Seely was appointed a transitman in the engineering corps of the Atchison, Topeka and Santa Fé Railroad Company, engaged in surveys in New Mexico under Lewis Kingman, M.

* Memoir prepared from information furnished by A. A. Robinson, M. Am. Soc. C. E.

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Am. Soc. C. E. In the fall of 1878 he was made Engineer of Track-Laying on the New Mexico and Southern Pacific Railroad Company, and served in that capacity until 1879, when he was ordered to Kansas to take charge, as Superintendent of Construction, of certain new lateral lines, which the rapid advance in the settlement of the State rendered necessary.

Under his direction and within a trifle over two years from the date of his appointment as Superintendent of Construction, the following lines were built and put in operation: Howard to Eureka, 29.2 miles; Wichita to Caldwell and Arkansas City, 91.9 miles; Burlingame to Manhattan, 56.6 miles; Florence to Ellinwood, by way of McPherson, 98.6 miles; Eldorado to Douglas, 24.3 miles; a total of 300.6 miles.

From the completion of the line from Burlingame to Manhattan until November 1st, 1880, Mr. Seely served as Superintendent in charge. On that date he was relieved of this duty and appointed Superintendent of Water Service of the main line and branches, his jurisdiction extending as far west as Pueblo and Las Vegas. These responsible duties he performed in addition to those of Superintendent of Construction until May 18th, 1881, when the water service was consolidated with the Bridge and Building Department.

In September, 1881, he was made Division Superintendent of the Las Vegas Division, extending from Raton to Wallace, a distance of 206.1 miles, and was also made Assistant Engineer, with direct charge of track, bridges, buildings and water service on his division.

In July, 1882, owing to ill health, Mr. Seely was obliged to resign these positions, and until October of the same year he sought by rest to regain his strength. On October 27th, 1882, he was made General Superintendent of the Sonora Railway, extending from a connection with the New Mexico and Arizona Railroad southward through Sonora to Guaymas, on the Gulf of California. On November 15th of the same year he was advanced to the position of Assistant General Manager of that line. By September, 1883, he was so weakened by consumption that he was forced to resign his position and leave Guaymas. He started for his old home at Oswego, Kendall County, Ill., but on October 2d, 1883, he died in his car just before the train reached Atchison. With him at the time were his wife and two doctors of the railway company's staff.

Mr. Seely was married in September, 1873, to Anzoletta E. Teller, of Oswego, who, with two sons, survived him. He was elected a Member of the American Society of Civil Engineers on February 1st, 1882.

He was never a strong man physically, but he possessed great powers of endurance, was devoted to his profession, and achieved much success in it, as shown by the preceding record.

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SAMUEL HENRY SHREVE, M. Am. Soc. C. E.*

DIED NOVEMBER 27TH, 1884.

Samuel Henry Shreve was born at Trenton, N. J., August 2d, 1829, his ancestors being among the colonial proprietors of New Jersey. He graduated from Princeton in 1848, and from the Harvard Law School two years later. He practiced law at Green Bay, Wis., and subsequently at Chicago until about 1853, when he returned East to prepare himself for the engineering profession, toward which he was attracted by a love of mathematics. His best work was done as an engineer and his reputation achieved as such.

He was engaged early in his career as an engineer in defining the complicated boundary lines under old colonial deeds in Ocean County, N. J., and made careful surveys for the purpose. Between 1860 and 1863 he was engaged as Engineer on the Southern Railroad of New Jersey and its branches, and was connected later with other surface railways. He was interested in the elevated railway projects in New York City from their inception in 1866, but took no active part until he aided the development of the Gilbert Elevated Railway just before it became the Metropolitan Elevated Railroad. He was retained as Consulting Engineer by one of the first rapid transit commissions in New York, and afterward by the Metropolitan Elevated Railroad. He became identified with the design and construction of the Sixth Avenue line and the structures on the east and west side of the city, built for the joint use of the New York and the Metropolitan Elevated railways. At this time, the most active in elevated railway construction, Mr. Shreve was recognized as a leader in this class of engineering work, and in 1881 was appointed Chief Engineer of the Brooklyn Elevated Railroad, a position he held to the time of his death, November 27th, 1884. All the essential features of the first Brooklyn elevated road were designed by him, and the most important portion of the line had been completed before he died.

In 1873 he published a work on the strength of bridges and roofs, which was translated into French. It was one of the first to succeed the pioneer work of Squire Whipple, printed in 1847, and discussed only the simple forms of trusses. It was to have been followed by another volume on the cantilever and the more complicated trusses, which was partly written at the time of his death.

The mathematical attainments of Mr. Shreve were notable, but in his published writings he has made use of only algebraic processes. In order to avoid the use of the calculus in problems where the maximum or minimum values of a function have to be determined,

* Memoir prepared from papers on file at the House of the Society.

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the equation containing only the first and second powers of the independent variable, he devised a process which he explained in an article in *Van Nostrand's Engineering Magazine*, Vol. XV, page 530. While recognizing the utility and convenience of graphics, he preferred algebraic methods of computation, in which he became so expert, that, having written an equation of the second degree, he effected the transformations mentally, and wrote down the values of the variable after a brief pause. He served as Associate Editor of the 1878 edition of "Johnson's Encyclopedia."

Mr. Shreve became a Member of the American Society of Civil Engineers, May 19th, 1869. In Volumes III and IV of *Transactions* will be found two discussions of some length written by him on the subject of arch trusses.

FREDERICK ELLSWORTH SICKELS, M. Am. Soc. C. E.*

DIED MARCH 9TH, 1895.

Frederick Ellsworth Sickels, famous as the inventor of the Sickels cut-off, a device by means of which the plans of James Watt for utilizing the economic advantages of expanding steam became for the first time thoroughly and practically applicable, and of many other no less ingenious if not equally important inventions, died of heart failure at Kansas City, Mo., at the age of 76 years, on March 9th, 1895. The modern factory, stationary, steam engine is substantially the work, in its evolution from the old Newcomen engine of 190 years ago, of three men: James Watt, who converted the machine of which Newcomen was the real inventor, from its primitive and enormously wasteful form into a comparatively efficient apparatus by securing reduction of internal wastes from about 95% to perhaps 30 or 40% by the adoption of a condenser separate from the working cylinder and by the use of a steam jacket, while improving at the same time its thermodynamic action, to such extent as the devices available at the time permitted, by the employment of a cut-off arrangement acting on a crude system of valve gearing; Frederick Sickels, who produced the first practicable drop cut-off gearing for the rotatory engine; and George Corliss, who devised a refined and specially contrived type of engine peculiarly adapted to the successful utilization of the ideas of Watt and of Sickels, and the highest refinement of steam-engine construction of the time. Watt was unable to avail himself fully of the advantages of his own plan of

* Memoir prepared by Robert H. Thurston, M. Am. Soc. C. E.

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expansion of steam by closure of the induction valve at an early point in the stroke of the piston, in consequence of two facts. Steam pressure was always too low in his engines to permit any considerable expansion, and even were the pressures higher, the rudeness of his devices and the ineffectiveness of his provisions for insuring steady rotation of the engine shaft precluded the employment of now familiar methods of expanding steam behind the piston. Sickels provided a system of construction and operation which permitted the detachment of the valve from its moving mechanism, and allowed it to drop back into its seat, thus almost instantly effecting closure at any desired point; while the use of a dash-pot containing water, oil, or air checked its fall as it approached its seat closely, and thus evaded the danger and annoyance consequent upon its unrestricted impact. Corliss perfected the engine later by adopting reduced clearances, partially balanced sliding valves moving with accelerated or retarded velocities close to the cylinder; a steam and an exhaust valve at each end, opening quickly and widely, detached automatically, as previously practiced by Sickels, dropping quickly, closing instantly, and moving comparatively little while closed. The invention of Sickels was an essential element of success in the steam-engine of the nineteenth century, and the name of Frederick Ellsworth Sickels rightfully stands beside that of James Watt, both as an inventor and as a builder, for he built numerous engines, stationary and marine, and gave a half-century of busy life to the work.

Sickels was a mechanic and an inventor by nature and inspiration. He was born in 1819 at or near Camden, N. J., received a common school education, and was then employed as rodman, at the age of 17, on the Harlem Railway, but promptly accepted the opportunity then offered him of an apprenticeship under the famous mechanic and engine-builder, James P. Allaire of New York. His father, John Sickels, who was an alumnus of Columbia College, later a physician and at one time Health Officer of New York, is said to have disapproved of his choice of a vocation, but the genius of the youth determined his future, and he completed his apprenticeship at the Allaire Works, and his became, as has been said by one of his biographers, "the only one of all the great names since James Watt to add a radically new and important elemental idea to the theory of steam-valve action, and to couple his new thought with mechanical details of such apt suitability as to bring it easily into the range of common practice,"* and this was before he had maintained his majority. The prime necessity for successfully putting into operation the idea of Watt and of producing a good expansion gear was ability to open wide the steam port, and to close it at the right moment in such manner as to cut off the steam supply instantly, giving a square corner on

* O. T. Porter.

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the indicator diagram and converting energy thermodynamically by as nearly adiabatic expansion as possible throughout the remainder of the piston stroke. This the Sickels drop cut-off accomplished actually and satisfactorily.

The invention of Sickels, for the first time in the history of the steam engine and in actual every-day practice, produced a distribution of steam approximating and closely coinciding with the ideal described by Watt in his patent of 1782, giving an indicator diagram with a sharp cut-off corner. This is one of the essential factors of maximum economy of operation of the engine. Its introduction immediately effected great improvement in current practice, and designing and constructing engineers have never ceased from that time to adhere to the principles and methods then first illustrated fully in application. Watt had proposed the expansion of steam by early cut-off, but he found, on attempting its introduction, what were insuperable difficulties in his time, in the irregular action of the engine, the impossibility of effecting prompt cut-off with his mechanisms, and the ignorance and obstinacy of the enginemen of the day, and practically gave up the task. He had adopted a ratio of expansion of two as being practicable, and advised four as the probably desirable figure. All his plans came to naught, however, in this field. Sickels overcame every difficulty by his ingenuity, enthusiasm and persistence, and led the way to the standard current practice of our time. His first patents were issued in 1841.

Immediately upon the public announcement of the new device, its introduction began. The firm of Thurston, Green & Company, then already well established and well known for its success in the construction of both stationary and marine engines of the then largest class, purchased the land rights, and Sickels himself, reserving the marine rights, went energetically about the introduction of the invention into the steam merchant and naval fleets. Thurston, Green & Company made the Sickels valve motion their exclusive form of steam valve gearing for mill engines, at once put the new engine on the market, and built many machines of all sizes and classes. Sickels was less prompt in securing a place for his invention on board ship, but he built it into the engine of the *Champion*, one of Vanderbilt's steamboats on Long Island Sound, and later applied it to many steamers on that Sound and on the Hudson River, commencing with the *North America*. The United States Navy, under the supervision of Engineer-in-Chief Haswell, adopted the cut-off, and it was employed on the *Waterwitch* in 1847, on the *Powhatan* in 1848, and subsequently on other vessels. It proved perfectly satisfactory on the side-wheel ships of that time, as on all slowly moving engines. On shore the invention found immediate and extensive application and was only driven out, about 1860, by the Corliss engine, patented in 1849, by

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George H. Corliss, in which the same result was accomplished by devices differing in form from those of Sickels, but always claimed by the latter and by many other engineers to be infringements upon his patents. This claim was passed upon by the Supreme Court of the United States after prolonged litigation, and the decision was rendered in favor of the defendant. No one questions, nevertheless, the claim of Sickels to the distinction of having originally and successfully led the way in the construction of a practicable and successful form of detachable valve gear, a drop cut-off.

Sickels made many other inventions, less well known simply because the greater light of the more important invention obscured that of the lesser ones. He never received full, or even moderate, compensation for the cut-off, or for any other of his characteristically ingenious and valuable devices. Perhaps the most remarkable of these minor devices, both in the eye of the inventor and in the estimation of others, was his steam-steering gear, an arrangement of small steam engines coupled to an ordinary steering gear by means of which a touch of the finger could be made to control the most powerful and speediest ship when in full career. This invention was first recognized publicly when exhibited at the London International Exhibition of 1862. Very powerful steamships were by that time afloat, and often required several men, sometimes eight or ten, at the helm, to steer the vessel readily and safely. The Sickels steam steering gear enabled one man to direct the course of the largest and fastest ship with as little difficulty as the smallest rowboat. A child might, with this device, put the helm of the *Campania* hard over when at full speed at sea. The arrangement consisted of a pair of small engines driving the barrel of the steering apparatus, as ordinarily attached to the steering wheel, and so placed that a small steering wheel, attached in such manner as to move the reversing mechanism of the engine, should make the same movements, in so doing, as would the large hand steering wheel. The helmsman operated this little wheel precisely as he was accustomed to handle the wheel of the old gear, and it required no greater effort than was needed to move the steam valve. All large vessels are now thus steered. By an ingenious device, the engines were made to move the helm over to the position desired by the wheelman, and to stop there automatically, following the hand of the steersman with absolutely perfect docility and precision.

Sickels patented altogether about thirty devices, many being improvements in detail upon his principal inventions. He lived and died, however, a "poor inventor," spending the comparatively small returns from his invaluable work, as fast as received, in the prosecution of experiments and in bringing out new devices. He deserved honors second only to those accorded James Watt and he met the fate of the

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prophet of the proverb who had honor except in his own country; but Sickels was little honored, even abroad. He lived and died almost unknown outside his profession, and so modest, retiring, and disinclined to urge his claims, that he was not extensively acquainted in his own guild. He spent his life mainly in the development and introduction of his inventions, assisted in legal matters by one of the ablest patent lawyers of the time, Mr. Edwin N. Dickinson, and in designing by a brother, Theophilus Sickels, M. Am. Soc. C. E., a well-known member of the engineering profession. After the expiration of the more important Sickels' patents, the brothers worked together in the construction of the Omaha Bridge across the Missouri River, and, later, the great inventor became the Chief Engineer of the Kansas City Water Works, which position he held at the time of his death.

Mr. Sickels' struggles to maintain his rights against infringers of his patents were among the most interesting events of the history of inventions. The suits against Corliss, particularly, brought into court the most famous legal and engineering talent of the time, and Curtis, Seward, Keller and Dickinson, as counsel, the Renwick brothers, Copeland, Hill, Greene, as experts, and the Thurstons, Benjamin F., famous later as the leading patent attorney and pleader of his generation, and Robert L., the founder of the first Providence Steam-Engine Company of 1837 and of the engine-building firms of Thurston, Gardner & Company and Thurston, Greene & Company, gave the controversy added interest by their presence on either side, by their able arguments and their learning, and by their knowledge of contemporary and earlier inventions. These cases were carried up to the Supreme Court, where after a series of victories in the Circuit Courts, Sickels was finally defeated. Corliss had, as adjudged by the court of last resort, for the first time combined the use of an expansion gear, original in idea with Watt, with a trip cut-off, original in principle with either Watt or some other inventor prior to Sickels, and with the attachment of the governor to determine the point of cut-off, original with Corliss himself in form, but in principle of earlier date, patented by Zachariah Allen, of Providence, as early as 1837, and used abroad in the form of the French cam before Corliss. Sickels was adjudged entitled to an exclusive right to his own particular form of trip and of dash-pot, and Corliss to his own particular group of details and method of governing; and the latter was declared not to infringe on the former. Many professionals were led to criticise this decision strongly, but it was subject to no appeal and ended the litigation.

Personally, Mr. Sickels was one of the most attractive and lovable of men. The writer came to know him in the early days of his patent litigations and retained an acquaintance with him throughout the remaining forty years of his life, and was always impressed by his kindliness, his patience, his good temper and his perfect integrity. He was

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careless in dress and in his habits of business, and curiously absent-minded; but he always adhered steadily to his plans, in spite of all discouragements, and found the way to his end with invariable certainty. He was a man of broad interests and extensive general information, and heartily loyal to his friends, absent or present. He was honest and true from first to last. He was never discouraged, and to the very end held fast to his faith in the ultimate success of his inventions, though without either credit or financial reward adequate to his work. And even when the award of the International Jury of the Centennial Exhibition, at Philadelphia, in 1876, made and signed by a jury composed of a group of the greatest men ever brought together on such an occasion, failed to see the light, he went about his work as placidly and cheerfully and determinedly as ever. The man was the equal of the inventor, the inventor had few equals in history, and few men have done more for the promotion of the development of civilization than did Frederick Ellsworth Sickels.

Mr. Sickels was elected a Member of the American Society of Civil Engineers on January 7th, 1891.

WILMON W. C. SITES, M. Am. Soc. C. E.*

DIED OCTOBER 1ST, 1885.

Wilmon W. C. Sites was born in 1849, and began his engineering career when he was nineteen years old as a rodman on the South Mountain Railroad. After spending about four months in this position, he entered the junior class of the Pennsylvania Polytechnic College, from which he was graduated in 1870.

From June, 1870, until February, 1871, he was rodman and assistant engineer on the construction of the Columbia and Port Deposit Railroad, a position he left to become Assistant Engineer of Surveys on the Stoney Creek branch of the North Pennsylvania Railroad. In a few months he resigned from this work and was appointed Assistant Engineer of the Berks County Railroad under Mr. J. Dutton Steele, and after the resignation of that gentleman, became Principal Assistant Engineer, a position he retained until the completion of the road in 1875.

In 1876 Mr. Sites removed to Jersey City, and commenced practice in general city engineering and surveying work, becoming Township Surveyor for West Hoboken and the Town of Union, in Hudson County, New Jersey, and superintending the construction of a number of public improvements in these places.

In 1877 he was appointed by Levi W. Post, M. Am. Soc. C. E., then Chief Engineer of the Jersey City Public Works Department, Surveyor

* Memoir prepared by E. W. Harrison, M. Am. Soc. C. E.

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in that department, having special charge of preparing a map of the water pipe systems of the city, proper knowledge of which was in a very confused state, the pipes having been laid at different periods by different municipal bodies, which in 1870 were united to form the present city, and no record having been preserved.

In 1879 a change of political control resulted in the appointment of a new chief engineer, and Mr. Sites returned to private practice.

In 1881 he was appointed Chief Engineer of the Public Works Department, and held the office until the spring of 1884. During this time he completed the second high-service distributing reservoir on Jersey City Heights, which had been commenced in 1871. Bringing this reservoir into use enabled him to draw down and clean the old reservoir, which had been in constant use for thirty years. He also laid the main supplying Bayonne City with water from the Jersey City works.

During Mr. Sites' term of office the deterioration of the water of the Passaic River at Belleville became a serious menace to the prosperity of Jersey City and Newark. The two cities combined in efforts to prevent the growth of the evil, and made an unsuccessful attempt to secure legislation to that end. Mr. Sites, as Chief Engineer of the Jersey City Department, took a very active part in the necessary investigations and preparations of evidence to sustain the cities' side of the agitation.

In 1884, on a change of administration, Mr. Sites resigned his office and entered into partnership with Edlow W. Harrison, M. Am. Soc. C. E., as engineers and surveyors.

The Legislature of 1884 had passed an act entirely revolutionizing the system of taxation of railroad and canal property in the State. The enforcement of the act was combated by all of the great transportation companies having interests in New Jersey. By the provisions of the act, which is now the settled system of taxation for this class of property in the State, the roads were required to be valued for assessment at their true value, and one of the elements of value was determined to be the cost of reproduction, less deterioration.

The new firm was selected by the State Board of Assessors, the body charged with the execution of this law, to act as the engineers for the State in the valuation of the railroad properties. In this work Mr. Sites was engaged until his death. The labor entailed was incessant. Not only was the field work of measurement of quantities and fixing market values of all the innumerable details which go to make up a railroad plant, and called for the supervision of a large corps of assistants, to be carried forward, but at the same time the attacks in the courts made by the companies upon the values as fixed and determined had to be met and answered. The examinations and cross-examinations of the members of the firm occupied many days. For over a year the work required fifteen or more hours out of the twenty-four.

Mr. Sites was never a healthy man, and, while Chief Engineer of

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the city, had shown symptoms of consumption. His perseverance was remarkable, and toward the end of his labor, he spent many days at active work, when another man would have been in bed.

Mr. Sites did not live to see the full completion of the work he had been engaged upon, and the establishment of its correctness by the highest courts of the State. Of him it may be truly said, "he died in harness," for within a week of his death, while lying upon the bed from which he never rose, he advised with his partner on the details of the engineering work of the cases then under consideration in the courts.

Mr. Sites was elected a Member of the American Society of Civil Engineers November 6th, 1878.

JOHN CHAMBERS THOMPSON, M. Am. Soc. C. E.*

DIED JANUARY 17TH, 1880.

John Chambers Thompson was the son of Jared and Jane Anthony, and was born in Philadelphia, Pa., February 5th, 1844. After the death of both parents, he was adopted by the Rev. Dr. Thompson, at that time preaching in Philadelphia, and his name was changed from Anthony to Thompson.

He was graduated from the Rensselaer Polytechnic Institute in the class of 1865, and was soon engaged as Assistant Superintendent of Construction of the Bessemer Steel Works of Messrs. Winslow, Griswold & Holley, of Troy, N. Y. He retained this position for two years, and then became Superintendent of the plant for about a year.

In 1868 he was appointed Assistant Engineer on the Croton Water-Works of New York City, and in 1870 held a similar position in the Department of Public Works of that city. In 1872 he again became Superintendent of the Bessemer Steel Works at Troy, and remained there for two years, when he went to the Crown Point Iron Company at Crown Point, N. Y., as Superintendent of its railroad department.

From 1877 to 1879 he was Assistant Engineer of the Croton Water-Works of New York City, and was subsequently connected for a short time with steel works at Cleveland, O.

His health then began to fail, and he went to Minnesota to recuperate. Consumption had attacked him, however, and he died of this disease on January 17th, 1880.

Mr. Thompson was a man very much beloved by those who knew him, of genial disposition and a thorough student. In the various positions held by him, he gave satisfaction to his employers, and his early death was a loss to the profession, in which, had his life been spared, he would no doubt have taken a prominent position.

He was elected a Member of the American Society of Civil Engineers on May 18th, 1870.

* Memoir prepared from a "Biographical Record of the Officers and Graduates of the Rensselaer Polytechnic Institute," by Professor H. B. Nason, and from information furnished by William H. Wiley, M. Am. Soc. C. E.

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CHARLES TRUESDELL, M. Am. Soc. C. E.*

DIED APRIL 23D, 1894.

In the death of Charles Truesdell, at Germantown, Pa., on April 23d, 1894, the American Society of Civil Engineers and the profession lost an old and useful member. He was born in Camillus, N. Y., in 1833, and educated in the academies of Onondaga and Madison counties. The son of an engineer, he early manifested a taste for the profession, to which he devoted forty years of his life.

In 1851, at the age of eighteen, he began his professional career under the late John McNair, in locating a railroad from Fort Niagara to Chippewa, on the Canadian frontier, and in the construction of the railroad from Lewiston to Niagara Falls. He was next associated with Hon. George Geddis in the service of the State of New York in surveys for the removal of the bar in Seneca River at Jack's Reefs. In 1853, under the late Hon. Van R. Richmond, he entered the service of the State of New York in enlarging the Erie Canal, subsequently having charge of some of the most important works on the canals, among them the new channel for Canandaigua River outlet, extensive dredging of Seneca River, the high embankments across the Montezuma marshes, and the Seneca River aqueduct. He continued in the service of the State many years in various capacities, and was appointed by the Governor to superintend the development of the salt wells at Montezuma.

He was Chief Engineer of the Cayuga Marsh Improvement Company, Chief Engineer in charge of surveys and location for a railroad from Chittenango to Cazenovia, N. Y., and was inspector in charge of the extension of harbor works at Cleveland, O., under the late General T. J. Cram and Major Walter McFarland, of the Corps of Engineers, U. S. Army. This position he resigned in 1868 to accept the appointment of Division Engineer in charge of the Uintah Division of the Union Pacific Railroad in Utah, continuing in that position until the road was completed in the fall of 1869. As Resident Engineer under Chief Engineer D. H. Wood, he had charge of the location and construction of the Montclair Railroad, now known as the Greenwood Lake Railroad, from Jersey City to Greenwood Lake, covering a period of about four years. At the expiration of this time he returned to the service of the State of New York, and was in charge of the construction of the State dam at the outlet of the Cazenovia Lake, of the completion of the Oneida Lake Canal, and of repairs to dams on Oswego River and Chenango Canal reservoirs and feeders. He was Chief Engineer for the

* Memoir prepared by Colonel George Truesdell, U. S. Army, and Major C. W. Raymond, M. Am. Soc. C. E.

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Syracuse Water Company, and subsequently Resident Engineer in charge of location and construction of the Delaware Division of the New York, Susquehanna and Western Railroad in New Jersey and Pennsylvania.

In 1891 Mr. Truesdell was appointed Assistant Engineer upon the extensive improvement of the harbor of Philadelphia under Major C. W. Raymond, M. Am. Soc. C. E. In this very responsible position he continued until his death, and he won the confidence, affection and respect of all his associates. He had a keen sense of responsibility and was earnestly devoted to duty, even while suffering from his last illness. Industrious and persevering to a remarkable degree, he was never contented until he discovered what he thought was the best solution of any problem, and his judgment was seldom at fault. He had a high sense of honor, was unselfish, kind and affectionate in all his family relations, and faithful in every relation of life.

In 1869 he married Mary Bradford, youngest daughter of Colonel John M. Fessenden, of Boston, Mass. Two children survive him, John Fessenden and Harriet T., wife of Carl Hering, an electrical engineer. He was elected a Member of the American Society of Civil Engineers September 15th, 1869.

LOUIS ROBERTS WALTON, M. Am. Soc. C. E.*

DIED NOVEMBER 9TH, 1885.

Louis Roberts Walton was born in Chester County, Pennsylvania, November 24th, 1842. He graduated from the Polytechnic College of the State of Pennsylvania in the class of 1863, and soon afterwards entered the employ of the Philadelphia and Erie Railroad Company, being connected with the harbor improvements at Erie, Pa. He remained with this company for four years, and then became Resident Engineer on construction of the Baltimore and Potomac Railway Company. In 1872 he was again employed on the Philadelphia and Erie Railroad, this time as Resident Engineer on construction, and the following year he went to the Pittsburg, St. Louis and Chicago Railroad Company as Principal Assistant Engineer.

In 1881 he accepted the position of Engineer of the St. Bernard Coal Company, of Earlington, Ky., the largest corporation of the kind in the State, and a pioneer in the introduction of improved mining machinery. He remained with this corporation until his death on November 9th, 1885. As an engineer he was careful, accurate and

* Memoir prepared by John B. Atkinson, M. Am. Soc. C. E.

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energetic. Coal-cutting machinery was just introduced into the mines at the time he became connected with the St. Bernard Company, and much of the success accomplished by the machines is due to his efforts. As a man and a citizen none stood higher. In his death the Society lost a most honorable member.

Mr. Walton was elected a Member of the American Society of Civil Engineers on April 1st, 1885.

ORLANDO BELINA WHEELER, M. Am. Soc. C. E.*

DIED JUNE 3D, 1896.

Orlando Belina Wheeler was born at Lodi, Mich., November 29th, 1835. He graduated from the University of Michigan in 1862, and then became acting assistant under Dr. F. Brünnow in the observatory of that institution. From 1863 to 1871 he was in charge of astronomical and geodetical triangulation parties on the United States Lake Survey, and from 1871 to 1878 was in charge of the computing division of the same survey. He made a specialty of astronomy, and while with the Lake Survey was sent as Assistant Astronomer by the United States Government to Siberia, to observe the transit of Venus in 1874, and to Colorado in 1878 on the total eclipse expedition of that year. In 1882 he was again sent by the Government as Assistant Astronomer with the expedition that went to Patagonia to observe the transit of Venus. Mr. Wheeler was the first to use the Morse alphabet in solar telegraphy, a method of communication he established while engaged in the triangulation of the Great Lakes.

After completing his astronomical work for the Government, he made a tour of the world, returning to this country in 1885 to become Principal Assistant Engineer in the office of the Missouri River Commission at St. Louis, Mo. This engagement continued until his death from apoplexy on June 3d, 1896, after an illness of but ten hours.

In social life Mr. Wheeler was gentle and unassuming, his quiet manner, quick intellect and learning making him beloved by all. He leaves a widow, one daughter and three sons.

Mr. Wheeler was elected a Member of the American Society of Civil Engineers on November 2d, 1887.

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* Memoir prepared from information furnished by B. H. Colby, M. Am. Soc. C. E., and from papers on file at the House of the Society.

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JOHN ALLSTON WILSON, M. Am. Soc. C. E.*

DIED JANUARY 19TH, 1896.

John Allston Wilson was born at Phœnixville, Chester County, Pa., April 24th, 1837. He was the eldest son of W. Hasell Wilson, Hon. M. Am. Soc. C. E., a native of Charleston, S. C., an eminent railroad engineer, and Jane Miller Wilson, both of whom are still living.

Mr. Wilson came of distinguished ancestry. He was the fifth in his generation to follow the profession of engineering. The name Allston was that of his great grandmother, who was the daughter of Capt. William Allston of Marion's brigade during the Revolutionary War, and a half-sister of Washington Allston, painter and poet. She was the wife of William Hasell Gibbes, a lineal descendant of Robert Gibbes, who was Chief Justice of South Carolina in 1708 and Governor in 1710-12.

John A. Wilson received his early education at private schools, and at the age of sixteen entered the Rensselaer Polytechnic Institute of Troy, N. Y., graduating with the degree of C. E. in July, 1856. In the early part of the year 1857 he was appointed topographer under Mr. John C. Trautwine on surveys in Central America for the Honduras Interoceanic Railway, on which work he continued until the return of the party to the United States in the summer of 1858. He then entered the service of the Pennsylvania Railroad Company as Assistant Engineer, and was engaged on work on the main line of that road. In 1860 he was promoted to the position of Principal Assistant Engineer, and until 1864 was engaged in the construction of railroad shops, bridges and branch railroads in the vicinity of Philadelphia. In addition to this, from 1861 to 1864 he was Chief Engineer of the Junction Railroad, connecting the Pennsylvania, the Philadelphia, Wilmington and Baltimore, and the Philadelphia and Reading Railroads, a piece of work involving varied and heavy construction. He also made the location for the Connecting Railway between the Pennsylvania Railroad and the Philadelphia and Trenton Railroad.

In 1863 Mr. Wilson served in the defence of his country as aide on the staff of General D. N. Couch, with the official rank of Captain. General Couch was then in command of the Department of the Susquehanna, and Captain Wilson had charge of the construction of fortifications in the vicinity of Harrisburg, Pa.

From 1864 until 1868 he occupied the position of Chief Engineer for the Pennsylvania Railroad Company, lessee of the Philadelphia and Erie Railroad, with headquarters at Williamsport, Pa.

* Memoir prepared by Jos. M. Wilson, M. Am. Soc. C. E.

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From 1868 to 1870 he was Chief Engineer of Maintenance of Way on the main line of the Pennsylvania Railroad, with residence at Altoona. During the period from 1870 to 1875 he made the location and superintended the construction of the Low Grade Division of the Allegheny Valley Railroad between Driftwood on the Philadelphia and Erie Railroad and Red Bank on the Allegheny River. At the same time he was also Chief Engineer of the Morrison's Cove Railroad, a branch of the main line of the Pennsylvania Railroad.

In January, 1876, the firm of Wilson Brothers & Company, Civil Engineers, Architects and Consulting Engineers, was organized, in which he was the senior member up to the time of his death. This firm has designed and had charge of the construction of many important structures, one of their recent great buildings being the Philadelphia and Reading Terminal Station at Philadelphia.*

One of the provisions of this firm allowed any member to hold an official position in his individual name, with the benefit of consultation and advice from the other members, all compensation being reported to and paid into the firm, and all such professional work being considered as part of its operations. Under this provision John A. Wilson acted personally in many important works as Chief or Consulting Engineer. Among these may be mentioned: Chief Engineer of the Bloomsburg and Sullivan Railroad, the North and West Branch Railroad, the Staten Island Rapid Transit Railroad, the Bellefonte and Buffalo Run Railroad, the Nittany Valley and Southwestern Railroad, the Columbia and Sullivan County Railroad, the Philadelphia Belt Line Railroad and the Philadelphia and Reading Terminal Railroad; Consulting Engineer of the Philadelphia and Reading Railroad for the Pennsylvania Avenue Subway, etc., etc.; also as Expert Engineer or witness for a large amount of railroad and similar public work, including many cases in litigation, such as railroad crossings, etc. Mr. Wilson displayed remarkable ability in matters connected with railroad law, making him extremely valuable as an expert adviser or witness in such cases. In some instances he worked up the whole plan of procedure for a case, and on an occasion not long before his death, when he rendered an opinion to a prominent lawyer, he was complimented by a letter from him, in which he said, "Your knowledge of the law on this subject should warrant your being made an LL.D."

In addition to his membership in this Society, dating from June 7th, 1876, Mr. Wilson was also a member of the American Institute of Mining Engineers, the Franklin Institute, the Historical Society of Pennsylvania, the Philadelphia Art Club and the St. Andrew's Society of Philadelphia. He was a vestryman and communicant of St. Andrew's Protestant Episcopal Church, West Philadelphia. He leaves a wife, four daughters and two sons.

* See *Transactions*, Vol. xxxiv, p. 116.

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JAMES HUGH STANWOOD, Assoc. M. Am. Soc. C. E.*

DIED MAY 24TH, 1896.

James Hugh Stanwood, who became an Associate Member of this Society October 3d, 1894, was born July 17th, 1860, at Brunswick, Me. His earlier education was received in the public schools of Portland. In 1879 he entered the office of Edward C. Jordan, M. Am. Soc. C. E., at Portland, and for about a year and a half was engaged in general land surveying. He afterward served on the survey for a railroad between North Bridgeton and Saccarappa, Me., and as assistant in the office of the City Engineer of Portland, and in the spring of 1881 went into the employ of the Maine Central Railroad Company as leveler and transitman.

Although in 1883 he entered the Massachusetts Institute of Technology as a student, and continued there for four years, he was still engaged during his summer vacations upon engineering work on the Maine Central Railroad. Graduating in 1887, he went to the Philadelphia Bridge Works as assistant to the designing engineer, and somewhat more than a year later returned to the Institute of Technology as Assistant in the Department of Civil Engineering. He afterward became Instructor, and continued in active service at the Institute, in charge of the drawing-room work in bridge design, until within a few months of his death, being compelled finally to give way before the inroads of Bright's disease.

He was enthusiastic in his attachment to the profession of civil engineering, and was not infrequently a contributor to its periodicals. He collected many specimens illustrating the rusting of iron and steel in exposed structures, such as bridges, made a careful study of the matter, and in 1895 presented a discussion on the subject before this Society. He also devised simple ormulas to accord with the experiments upon the strength of wooden posts, which appeared in the *Railroad Gazette* in 1892 and 1894 and have been well received by engineers.

Mr. Stanwood was characterized by great faithfulness to all the details of his duties, and by an abounding fund of good nature which, combined with a frank and generous spirit, won for him many friends and the enduring regard of all his students. He died May 24th, 1896, in his thirty-sixth year, leaving a widow and three children.

* Memoir prepared by Dwight Porter, M. Am. Soc. C. E.

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CHARLES WOOD, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 28TH, 1895.

Charles Wood was born in Edinburgh, Scotland, January 8th, 1862. He was educated in the Academy of that city. It was his early ambition to obtain a position in the Royal Engineers, but circumstances changed his plans. To perfect his knowledge of the French language, he studied a year in France. This was about 1878. In 1881 he came to America. In April of that year he found employment in the mechanical drawing-room of the Edge Moor Iron Works, where he was engaged until June, 1882. Doubtless his experience there brought him to realize the importance of a more thorough technical education as a preparation for his chosen profession, and, in the autumn of 1882, he entered the Massachusetts Institute of Technology, from which he graduated in 1886, with high rank in his class. The subject of his thesis was a design for a cantilever bridge over the St. Lawrence River at Lachine, P. Q. To familiarize himself with the requirements of the problem, he spent a vacation at Lachine.

There is abundant testimony that while at the Institute he was an earnest and faithful student, a leader among his fellows, and that he was held in the highest esteem by both faculty and students. During the vacation in 1884, he was employed in the bridge erection force of the Chicago, Milwaukee and St. Paul Railroad.

On the completion of his course at the Institute, he was at once offered a position in the drafting-room of the Boston Bridge Works, where he remained until October of the same year, at which time he accepted a position in the drafting-room of the Keystone Bridge Company. From January to May, 1887, he was engaged on special drawings for the terminals of the Minnesota and Northwestern Railroad at St. Paul, Minn. In May, 1887, he accepted a position with the Pittsburg Testing Laboratory, to inspect shop work at Athens, Pa., for the Poughkeepsie Bridge, inspecting also bridge-work at the Union Bridge Company's shops at Rochester, N. Y., and bridge material at the Elmira Rolling Mill. On January 15th, 1888, he entered upon an engagement as Assistant Engineer on the Cincinnati, Hamilton and Dayton Railroad, Joseph Ramsey, Jr., M. Soc. C. E., being Chief Engineer of the road. Upon Mr. Ramsey's retirement, to accept the position of Assistant to the President of the Cleveland, Cincinnati, Chicago and St. Louis Railroad, Mr. Wood became Acting Chief Engineer, and, in August, 1891, was appointed Chief Engineer of the Cincinnati,

* Memoir prepared by S. Whinery, M. Am. Soc. C. E.

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Hamilton and Dayton Railroad system, a position he held with ability and honor until his death.

During this engagement extensive improvements were made on the road, embracing bridge renewals, the erection of an iron freight house at Cincinnati, and the construction of a large modern grain elevator at Toledo, O. The maintenance of way department was also under Mr. Wood's charge. During this time he resided in the village of Wyoming, a suburb of Cincinnati, and was one of three commissioners, Mr. Ramsey being another, appointed to design and construct a system of water supply for the village. After Mr. Ramsey removed to St. Louis, the mechanical and technical work of the commission fell largely to Mr. Wood, and to him was due in no small measure the practical success of the enterprise. He was re-elected to the position, which he held at the time of his death.

In the early part of the summer of 1895 Mr. Wood was taken sick with what his physician thought to be some disorder of the digestive system, and he was forced to discontinue all work for a time. Later in the season he was able to be at his office a few hours nearly every day, but there was no permanent improvement in his condition. As autumn came on he was confined to his home. Later his physician surmised that his malady was of a more serious nature than had been suspected, and a surgical operation, which was deemed advisable, disclosed that he was a victim of cancer of the stomach. He did not rally from the operation, and died on November 28th, 1895.

Mr. Wood became an Associate Member of the American Society of Civil Engineers July 4th, 1894. He was also a member of the Engineers' Club of Cincinnati, to which he contributed valuable papers. He took great interest in the American Society of Civil Engineers, and before his death collected notes for a paper for the Society on "The Construction of Grain Elevators." It is to be greatly regretted that he did not live to complete this paper, as it would have been of great practical value to the profession.

Mr. Wood was especially fond of structural work, and, while able and efficient in other departments, he developed unusual aptitude and talent in designing bridges and other structures. He was an energetic and tireless worker, who loved his profession next to his family and home. His friends have good reason to believe that had he not been cut off in his early manhood, he would have attained a front rank in his profession. In his business intercourse he was affable, courteous, considerate and just, and, at the same time, outspoken and decisive.

Letters to the writer since Mr. Wood's death, from instructors and fellow students at college, and from business and professional associates since his graduation, all bear strong testimony to his ability and sterling character. Mr. M. D. Woodford, President of the Cin-

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cinnati, Hamilton and Dayton Railway Company, in a letter, dated January 13th, 1896, pays this high tribute to him as a man and a civil engineer:

"In the death of Mr. Wood, which was greatly deplored, the Cincinnati, Hamilton and Dayton Railway has lost a valuable official, whose eminent personal and professional qualities merited my most sincere appreciation.

"Mr. Wood commenced his service with the Cincinnati, Hamilton and Dayton Railway in January, 1888, as Assistant Engineer, which position he filled so creditably that in January, 1890, he was promoted to the position of Acting Chief Engineer, which afforded him an opportunity to demonstrate his ability, which was soon recognized by his appointment in August, 1891, to the office of Chief Engineer.

"Being well equipped by education and experience, he performed the duties of that office with marked success. Proficient in all branches of his profession, his forte was bridge engineering. He was also an expert in structural work. During the eight years of his connection with the Cincinnati, Hamilton and Dayton Railway Company much exceptionally important work was done under his supervision, which bears testimony to his excellent judgment and qualifications as an engineer, notably the planning and construction of our extensive Ivorydale yards, the erection of new freight houses at Cincinnati, the construction of a large grain elevator at Toledo, O., and the building of bridges over Silver Creek, Big Williams Creek and the Miami River.

"In his personal life he was esteemed and beloved for his splendid qualities of mind and heart, for his conscientious sense of duty and the high principles by which he was ever guided. Affable and courteous in his manners, his relations with all with whom he was brought in contact were at all times cordial and pleasant. Owing to his extreme modesty and his aversion to putting himself forward, only those who knew him intimately or were closely associated with him thoroughly appreciated his true value. Cut off by the Dread Reaper so early in his career, it is sad to think that a life of great usefulness and promise is ended just in its prime."

Mr. Wood's professional and business associates uniformly speak in the highest terms of praise of his personal and professional honor, his integrity, reliability and conscientious character. In his personal and social relations he was respected by all, and esteemed and beloved by his many friends for his sterling qualities of mind and heart.

Mr. Wood was married in May, 1890, to Miss Ruth Cowing, of Wyoming, O., who, with two children, survive to mourn the untimely loss of a most devoted husband and father.

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FRANK BERESFORD, Jun. Am. Soc. C. E.*

DIED DECEMBER 12TH, 1887.

Frank Beresford was born at Cincinnati, O., on April 20th, 1861, and resided in that city for most of his life. He graduated from the civil engineering department of the University of Cincinnati in 1884, and immediately entered the engineer's office of the Cincinnati, New Orleans and Texas Pacific Railroad Company. His stay there lasted but a few months, however, for he entered the service of the Cincinnati, Hamilton and Dayton Railroad Company in September of that year, where he remained until his death.

His progress in his profession was quite rapid, for in the course of a year he rose from the position of Draftsman to that of Principal Assistant Engineer, with charge of the drafting, field work, bridge designing and inspecting. He was engaged in the discharge of these duties at the time of his death, which occurred on December 12th, 1887. His attainments as an engineer were exceptional for a man of his years, and his high character won him many friends. An intimate acquaintance, Joseph Ramsey, Jr., M. Am. Soc. C. E., writes as follows concerning him :

"In my opinion, his death was a great loss to the engineering profession, as I considered him one of the brightest and most promising engineers I ever had any connection with. He was a thorough-going, efficient and capable man, and had one of the most upright, honorable characters of any of my acquaintances."

Mr. Beresford was elected a Junior of the American Society of Civil Engineers on September 7th, 1887.

WILLIAM ALEXIS GEORGE EMONTS, Jun. Am. Soc. C. E.†

DIED NOVEMBER 5TH, 1887.

William Alexis George Emonts was born in 1847, at Spire, Bavaria, and was educated in the gymnasium at that place. He served as a lieutenant in the Bavarian army in the wars of 1866 and 1870–71, and during this time studied and practiced military engineering somewhat.

Soon after the close of the Franco-German war he came to this country, and in February, 1873, became an instrument man in the Engineer-

* Memoir prepared from papers on file at the House of the Society.

† Memoir prepared from information furnished by D. McN. Stauffer, Theodore Voorhees and William Hunter, Members Am. Soc. C. E., and from papers on file at the House of the Society.

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ing Corps of the Philadelphia and Reading Railroad. In the fall of 1874 he became assistant to Mr. Henry W. Potts, Division Engineer on the Philadelphia and New York New Line Railroad, and retained this position until the completion of construction in June, 1876. He then became Assistant Engineer and Draftsman on the North Pennsylvania Railroad. In 1876 Mr. Emonts went to Germany and in 1879 to Central America.

During 1882 and 1883 he was engaged on the construction of the Shamokin, Sunbury and Lewisburg Railroad, and was located at Sunbury. During the latter part of 1883, he went to Philadelphia and was engaged on the surveys of the Schuylkill River East Side Railroad, until the conclusion of that work, some time in 1884. About 1885 he published a small pamphlet containing metric conversion tables.

Mr. Emonts was elected a Junior of the American Society of Civil Engineers on September 6th, 1876.

VERNON HILL GRIDLEY, Jun. Am. Soc. C. E.*

DIED SEPTEMBER 17TH, 1896.

Vernon Hill Gridley was born on a farm near Owosso, Mich., on February 11th, 1867. His early education was received in the country schools most convenient to his home. When he was twelve years of age the family left Michigan and settled in Monroe County, New York. From 1885 to 1887 he attended the Normal School at Brockport, and during the winter of 1887–88 he took the course in the Rochester Business University. The following year he had charge of the commercial department of a college or high school in Toronto, Canada, and after serving one year there he assumed a similar position in Fairfield Academy, Herkimer County, New York. In September, 1890, he entered the Rensselaer Polytechnic Institute, from which he was graduated with the degree of Bachelor of Science in 1893. He remained another year at the Institute, assisting Prof. Wm. P. Mason with his classes in chemistry, and in June, 1894, received the degree of Civil Engineer.

The first work which he did after leaving Troy was the taking of observations and the determination of the flow of water at the outlet of Lake George at Ticonderoga. In September he was appointed a leveler in the Department of City Works, Brooklyn, N. Y., and was employed in running a series of levels to determine the actual elevations of all the improved streets in the city. Early in 1895 he

* Memoir prepared by N. P. Lewis, M. Am. Soc. C. E.

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entered a competitive examination for assistant engineers in that department, in which he obtained high rank, and was at once appointed. From then until the time of his death he was engaged upon the construction of street pavements in the city of Brooklyn.

He was elected a Junior of the American Society of Civil Engineers on February 4th, 1896.

He contracted typhoid fever in August, 1896, and died at St. Mary's Hospital, Brooklyn, on September 17th, 1896.

Mr. Gridley's brief professional career gave promise of a bright future. Quiet in manner and reserved in disposition, he impressed all who came in contact with him as a man of force and character. His industry, integrity and capability would have insured success in any field. His thoroughness in all matters of detail and his marked business capacity rendered his services especially valuable, both in designing and executing engineering work, while his admirable personal qualities won him many friends, who have learned of his untimely death with the keenest regret.

RUSSELL WADSWORTH HILDRETH, Jun. Am. Soc. C. E.*

DIED DECEMBER 23D, 1895.

Russell Wadsworth Hildreth was born in New York City on February 12th, 1865. He entered the School of Mines at Columbia College in 1881, and was graduated in June, 1885, with the degree of Engineer of Mines. Four months later he became a draftsman in the New York office of George S. Morison, Past-President Am. Soc. C. E., remaining there until March 1st, 1886, when he went to the Buffalo shops of the Union Bridge Company. There he was engaged in the inspection of the double-track and highway bridge across the Missouri River at Omaha, Neb., for the Union Pacific Railway Company, and two bridges for the Oregon Railway and Navigation Company. From April 1st to August 1st, 1887, he was Assistant Engineer on the Union Pacific Bridge at Omaha, and then went to the Athens shops of the Union Bridge Company to take charge of the inspection of a bridge across the Willamette River at Portland, Ore., for the Oregon Railway and Navigation Company.

On January 1st, 1886, the firm of Hildreth and Nettleton, inspectors of bridges, was formed by Mr. Hildreth and Mr. W. A. Nettleton. On September 1st, 1888, this firm became R. W. Hildreth and Company, owing to the withdrawal of Mr. Nettleton and the association of Mr.

* Memoir prepared from information furnished by Mr. P. S. Hildreth and from papers on file at the House of the Society.

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P. S. Hildreth. Mr. Hildreth was connected with it up to the time of his death.

The work carried on embraced the inspection of many well-known bridge structures during manufacture and erection, including the New London and Red Rock bridges, general work for many of the principal cities, railroads and other corporations, and the preparation and examination of plans for steel work and the inspection of existing structures.

Mr. Hildreth was elected a Junior of the American Society of Civil Engineers on January 4th, 1888. He was a member of the American Society of Mechanical Engineers, the American Institute of Mining Engineers, and the Engineers' Club of New York. He died of typhoid fever and heart failure, after a brief illness.

ALBERT JACOB STAHLBERG, Jun. Am. Soc. C. E.*

DIED AUGUST 19TH, 1887.

Albert Jacob Stahlberg was born in Denmark in 1846. He was graduated in 1868 from the Polytechnic Institute (*Landbohöiskolen*) in Copenhagen as a "Forest Officer," and served in Sweden in 1868 and 1869 as Assistant on the work of regulating about 5 000 acres of woodland, which included the surveying and valuation of the property and the improvement of the streams running through it.

Late in 1869 he came to this country, and soon found employment as Assistant to Mr. George Beckwith, at that time the City Engineer of Bridgeport, Conn. In the summer of 1870, he entered the office of Messrs. Welton and Bonnett, of Waterbury, Conn., and under their direction was employed on the construction of water-works, railways and city works of various kinds. He remained in this office a number of years and then went to the Pacific Coast. For a time he was Assistant City Engineer of Los Angeles, Cal., and afterwards was connected at different times with the South Pacific Coast Railroad, the Oregonian Pacific Railway, and the Oregon and California Railroad.

In 1885 he visited Copenhagen on account of failing health, and remained in Denmark until his death on August 19th, 1887.

Mr. Stahlberg was elected a Junior of the American Society of Civil Engineers on March 4th, 1874.

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* Memoir prepared from papers on file at the House of the Society.

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WILLIAM HOWLAND ASPINWALL, F. Am. Soc. C. E.

DIED JANUARY 18TH, 1875.

William Howland Aspinwall, the founder of the Panama Railroad and Pacific Mail Steamship Companies, was the grandson of Captain John Aspinwall, one of the most prominent shipmasters of the New York merchant marine before the Revolution, and the son of John Aspinwall, a well-known merchant of that city in the early part of this century. He was born in New York on December 16th, 1807, and was educated in local private schools. At an early age he became a clerk in the mercantile house of G. G. & S. S. Howland, his uncles, and advanced so rapidly that in 1832 he was taken into partnership. About five years later Gardner and Samuel Howland withdrew from the active management of the affairs of the house, and its name was changed to Howland & Aspinwall. The business was very extensive, particularly with countries on the Pacific and Mediterranean coasts, the East and West Indies, and England, and the firm owned at one time over fifteen ships, including several Liverpool packets.

It was this wide range of business relations that led Mr. Aspinwall to appreciate the importance of the Panama route. California had just been annexed to the United States, and he believed that better means of communication across the Isthmus of Darien would prove profitable to those furnishing it, as well as a great aid in developing American commerce. Congress had sold contracts for carrying mail by steamers from Chagres to New York and New Orleans, and from Panama to San Francisco, but the persons who obtained them were unable to carry out their provisions. Finally George Law, the New York street railway builder, bought the Atlantic line, and Mr. Aspinwall obtained control of that on the Pacific. The former was then considered a good investment, but Mr. Aspinwall's commercial acquaintances regarded his purchase as a very poor bargain. It was merely part of a great undertaking, however, which developed under his management into a very profitable business, of great importance to commercial interests. In 1848 the Pacific Mail Steamship Company was chartered in New York State to carry out the mail steamer contracts, and Mr. Aspinwall remained its president until his retirement from active business in 1856.

This company carried out but half his plans, however, for he recognized, as soon as the rush for California began, after the discovery of gold on the Pacific Coast, that some method of solving the problem of rapid transit across the Isthmus, given up as hopeless by early English and French investigators, must be devised. His idea was to build

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a railroad across the narrow ridge of land. He found in John L. Stevens an associate familiar with the country who was willing to make an exploration of the route in company with an engineer. The result of this investigation showed that such a line could be built, and Messrs. Aspinwall, Stevens and Henry Chauncey made a contract with the authorities of New Grenada to build a railroad. The Panama Railroad Company was incorporated under the laws of the State of New York to take over these obligations, and Mr. Stevens was elected president. A contract was made with George M. Totten and John C. Trautwine for the construction of the line, and work was begun in May, 1850. It progressed slowly on account of the great natural difficulties of topography and climate, but was finally opened early in 1855. Its Atlantic terminus on Navy Bay was an uninhabited spot at the time the railroad was projected, and was for some time overshadowed in size and commercial importance by the neighboring town of Chagres. As the railroad progressed, however, it became an important place and was named Aspinwall, after the man whose enterprise had developed that region. This name was employed by English-speaking people until a few years ago, when the Colombian authorities refused to transmit mail matter to it unless addressed Colon, the Spanish name of the place. In the first seven years of operation of the road, the net earnings were nearly $6 000 000, a good proof of its founder's business sagacity. As a mark of appreciation of the work of the three projectors of the line, Messrs. Aspinwall, Stevens and Chauncey, the directors of the company erected, a few years before Mr. Aspinwall's death, a large monument at Colon, on the base of which their busts were carved. This monument stands on the beach in front of the company's property, and is one of the prominent features of the town.

After these great companies were placed on a thoroughly sound basis Mr. Aspinwall practically retired from business. Much of his time was spent in travel, and he took particular pleasure in collecting pictures and in improving his country estate near Tarrytown.

Mr. Aspinwall was elected a Fellow of the American Society of Civil Engineers on July 9th, 1870. He died on January 18th, 1875.

GEORGE WASHINGTON CASS, Jr., F. Am. Soc. C. E.*

DIED MAY 21ST, 1888.

George Washington Cass, Jr., was born on his father's lands, near Dresden, Muskingum County, Ohio, on March 12th, 1810. He was the oldest child of George Washington Cass and Sophia Lord, and

*Memoir prepared by Cass K. Shelby, Esq.

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grandson of Jonathan Cass, a Captain of volunteers in the Continental Army, and afterwards a Major in the U. S. Army. Jonathan Cass lived in Exeter, N. H., and moved with his wife, Mary Gilman, and family in 1800 to Ohio, where he settled finally on the Muskingum River near Dresden. Owing to the schools in that new region being of the most elementary character, George W. Cass, Jr., was sent to Detroit, Mich., in 1824 for the purpose of being educated at the Detroit Academy, then under the charge of Rev. Ashbel Wells, and while there he lived with his uncle, Lewis Cass, at that time Governor of Michigan Territory. Four years later he obtained an appointment from his native state as a cadet at the U. S. Military Academy at West Point, and graduated from there in June, 1832, at the head of his class in the principal studies, and among the first five in general academic studies.

Instead of receiving the usual two months' leave of absence on graduating, he was at once ordered to report to General Scott, then in New York organizing an army to proceed against the Indians, who were collected in large numbers in the Northwest under Black Hawk. Although having only the rank of cadet, he was placed in command of a company of infantry just recruited into service, and assigned to that portion of the army under General Twiggs. On the way to the frontier the command of General Twiggs was so much reduced in numbers by the Asiatic cholera that a number of companies were broken up for the purpose of filling others to a proper complement, and thus, the number of officers being in excess of the demand for service, Capt. Cass was transferred to the Department of Topographical Engineers. He served six months in this department, and was then transferred to the Department of Military Engineers, in which he remained until October, 1836, when, resigning his position, he received from President Jackson an appointment of Civil Engineer on the National Road, in which capacity he continued until the completion of that road in the states of Virginia, Maryland and Pennsylvania. During this service Capt. Cass erected over Durlap's Creek, a tributary to the Monongahela River, the first cast-iron bridge ever built in the United States.

He was an early and persistent advocate of the improvements of the Monongahela River by locks and dams, and contributed to the procuring of the charter and organization of a company. As its engineer, he made the survey and located and superintended the construction of locks Nos. 3 and 4. After the suspension of the work by the inability of the State of Pennsylvania to pay its appropriation, and the sale of the State stock to private parties, he was a member of the Board of Managers, and was actively instrumental in organizing a company from the new shareholders and the framing of a contract which insured the completion of the work in 1844.

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On the completion of the Monongahela improvements to Brownsville, Pa., he organized the first steamboat line on that river, and also the first fast transportation lines across the mountains by relays of teams similar to stage lines, thus building up a large carrying trade between the East and the West *via* the Monongahela River and Pittsburg.

In 1849 he established the Adams Express across the mountains from Baltimore, effected the consolidation of all the Adams Express lines between Boston and St. Louis, and south to Richmond in 1854, and in the year following was elected President of the consolidated company, with offices in Pittsburg, Pa.

In January, 1856, he was elected President of the Ohio and Pennsylvania Railroad Company, then possessing a road completed to Crestline. This line was finally extended to Pittsburg and Chicago, and became the Pittsburg, Fort Wayne and Chicago Railway. Mr. Cass remained at the head of this company for twenty-five years. In 1859 he was appointed a member of the Board of Visitors to the U. S. Military Academy. He was also president of the Continental Improvement Company of Pittsburg which built the Grand Rapids and Indiana Railroad.

Mr. Cass was one of the Commissioners named by Congress to organize the Union Pacific Railroad Company, and was a member of the first Board of Directors of that company. He declined the office of treasurer and president. He also became a member of the Smith syndicate which took possession of the franchise and debts of the Northern Pacific Railroad Company in 1866, and became greatly interested in the development of the Northwest. When the Northern Pacific Railroad was constructed and put into operation, he became its president on October 1st, 1872, and transferred his residence from Pittsburg to New York City. He was at this time also President of the Southern Railway Security Company which controlled the properties of the East Tennessee, Virginia and Georgia Railroad and other southern lines. While connected with the Northern Pacific system Mr. Cass bought a tract of land seventeen miles west of Fargo, No. Dak., having in view the cultivation of wheat in that region. At the same time Mr. Charles P. Cheney, of Boston, bought lands adjoining, and jointly they engaged the services of Oliver Dalrymple to superintend the united estate, which became known as the famous Cass-Cheney farm. Mr. Cass resigned his office of president of the Northern Pacific Railroad in 1875 to take the receivership of that company, and after the reorganization in August following he went to Europe for a rest. With the exception of a visit home the next year, he remained abroad until the spring of 1881.

Mr. Cass did not again engage in active business after severing his connection with the Northern Pacific. He continued to live in New York City, however, where he died March 21st, 1888.

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George W. Cass married first, January 5th, 1835, Louisa Smith, second daughter of George and Mary (Kennedy) Dawson, of Brownsville, Pa., and had by her one child, a daughter. His wife died at Dresden, O., seven years after, and he married secondly, September 14th, 1843, Ellen, the third daughter of George Dawson, of Brownsville, and had by her five sons and six daughters.

He was elected a Fellow of the American Society of Civil Engineers on March 30th, 1871.

THOMAS C. DURANT, F. Am. Soc. C. E.*

DIED OCTOBER 5TH, 1885.

Thomas C. Durant was born in Lee, Berkshire County, Mass., in 1820, and came of a family which had played a prominent part in the early history of that region. He studied medicine at the Albany Medical School and practiced for about three years, but finally gave up the profession to enter upon the business career in which he subsequently achieved remarkable success. He became a member of the Albany shipping firm of Durant, Lathrop & Company, which had branches in New York, Chicago and Boston, and carried on a large business. Dr. Durant was the head of the New York branch. He was the owner of a number of vessels, leased many more, and his European trade, particularly in wheat, was very profitable up to about the time of the French Revolution of 1848.

The shipping interests with which he was connected led him to recognize the possibilities of the West, and made him a strong advocate of internal improvements which would open up the resources of that region. He was actively engaged in promoting the interests of the Michigan Southern Railroad, and was one of the leading contractors for the construction of the Peoria and Bureau Valley and the Mississippi and Missouri Railroads. He made surveys for a railway line in the Platte Valley several years before the organization of the Union Pacific Company, and in 1863 had surveys made at his own expense from Omaha to the basin of Great Salt Lake. In 1863 he took an active part in raising the $2 000 000 stock subscription required by Congress before the Union Pacific Company could be organized. His active interest in the work continued until the last rail was laid, the financiering of the great undertaking receiving most of his attention.

As soon as this line was finished Dr. Durant began building the Adirondack Railroad, of which he was the chief stockholder, President and General Manager at the time of his death.

He was elected a Fellow of the American Society of Civil Engineers on November 18th, 1870.

* Memoir prepared in the office of the American Society of Civil Engineers.

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SIDNEY DILLON, F. Am. Soc. C. E.*

DIED JUNE 9TH, 1892.

Sidney Dillon, the well-known financier and contractor for railroads having a total length variously estimated at 2 500 to 3 000 miles and costing from $75 000 000 to $100 000 000, was a farmer's boy in early life. He was born on May 7th, 1812, in Northampton, Montgomery County, N. Y., where the family had resided for several generations. His grandfather was a Revolutionary soldier, and was present at the surrender of Burgoyne. Young Dillon was brought up like most boys in similar conditions at the time, receiving as good an education as the local schools afforded.

His railroad career began as an errand boy on the Mohawk and Hudson Railroad from Albany to Schenectady, and when this road was finished, he was employed in a similar capacity on the Rensselaer and Saratoga Railroad. Later, he became an overseer for Crane & Clark, who had a contract on the Boston and Providence Railroad near Sharon, and remained with them for two years, until the contract was completed. Then he was made a foreman for the same firm on its contracts on the Stonington road in Connecticut and on heavy rock work near Charlton, Mass., on the Western Railway, now part of the Boston and Albany line. In the latter position he had the good fortune to become intimately acquainted with Capt. W. H. Swift, at that time the engineer in charge of work. Capt. Swift took a strong interest in the young man, and when it came time to put in bids for new work urged him to make tenders on his own account. Mr. Dillon had little money at the time, but he thought he saw his way clear to undertaking a small section near Hinsdale. His bid was accepted and the work was completed satisfactorily in 1840.

This was the beginning of a contracting career of unusual extent and success. Mr. Dillon is described by those who were acquainted with him about this time as an unusually fine leader of men. Over 6 ft. tall, heavily built yet active, and speaking in a direct, incisive manner, he conveyed the impression of a man who knew what to do and how to do it, and was fully able to command others in carrying out his plans. His second contract was on the Troy and Schenectady Railway, where he took some heavy work in clay about 2 miles from Troy, which was carried out by means of a steam shovel. Then he became a member of the firm of Boody, Ross & Dillon, which built 6 miles of the Cheshire Railroad in Vermont and the Hartford and Springfield Railway, 26 miles long, part of the payment of the latter being made in stock. At this time he also had an individual contract to construct

*Memoir prepared in the office of the American Society of Civil Engineers.

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6 miles of difficult line on the Vermont and Massachusetts Railroad. As a partner in the firm of Dillon & Pratt he was interested in the construction of 7 miles of the Rutland and Burlington Railroad near Burlington, Vt. A little later Boody, Ross & Dillon built the Central Railroad of New Jersey from Whitehouse to Easton, a distance of 29 miles, taking their pay entirely in stocks and bonds. The work was difficult, but was finished in two years by means of steam shovels. The firm was also engaged about the same time in widening 20 miles of the Morris Canal, and a little later contracted to build the Boston and New York Central road. The latter company failed after 30 miles were built, and the contractors were forced to attach and sell the rolling stock and other visible assets to obtain part of the money due them. Afterwards the company built 100 miles of the Philadelphia and Erie road west from Lock Haven ; and, somewhat later, Mr. Dillon was interested, as a member of the firm of Dillon, Clyde & Chapman, in contracts on the Erie and Cleveland, the New Jersey Central and the Mooris and Essex Railroads.

In 1865 he began his active association with the Union Pacific Railway, and during the next four years his wide experience as a railroad builder was of much value to the men in charge of that great enterprise. He was present at the ceremony of laying the last rail in 1869, and the silver spike with which the road was completed remained in his possession until his death. During this time he was also interested with John I. Blair in Iowa railroads, and had a 70-mile contract on the construction of the Boston, Hartford and Erie Railroad, which, however, was not completed on account of the lack of funds of the railroad company. A little later he had contracts on the Connecticut Valley, the Chillicothe, Council Bluffs and Omaha, and the Canada Southern roads, built the Paterson branch of the Morris and Essex line and was engaged on many smaller works.

All these contracts brought Mr. Dillon a large fortune, which was principally invested in railroad securities. The management of these investments gradually occupied a larger part of his time, and after 1870 he was known chiefly as a financier. He was a director of the Union Pacific Company for many years and its president during a considerable part of this time. He was associated with Jay Gould in the management of many of the properties controlled by the latter, and was a director in the Western Union Telegraph, the Manhattan Elevated Railroad, the Missouri Pacific Railway, the Pacific Mail Steamship, the Chicago, Rock Island and Pacific Railway, the Wabash Railway, the Canadian Southern Railway, the Wheeling and Lake Erie Railroad Companies and many smaller organizations.

Mr Dillon was elected a Fellow of the American Society of Civil Engineers on March 26th, 1870. He was married in 1841, and left two daughters, four grandsons and one granddaughter.

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HENRY FARNAM, F. Am. Soc. C. E.*

DIED OCTOBER 4TH, 1883.

The career of Henry Farnam, prominent as an engineer, contractor and railroad president during the building up of the Central States, is remarkable for the fact that for nearly thirty years his work was of a routine nature, and for most of the time connected with a more or less financially unsuccessful undertaking. This long period of apprenticeship, if it may be called such, brought its fruits, however, in thirteen years of unusually successful operations on a great scale, yielding pecuniary returns commensurate with their importance, and enabling Mr. Farnam to spend the last twenty years of his life in a retirement marked by many gracious acts.

He came of a Connecticut family that settled in Scipio, Cayuga County, N. Y., late in the last century. The country was then practically a wilderness, and the settlers were pioneers in many respects. Henry Farnam was born November 9th, 1803, and soon displayed an unusual fondness for books and a lack of interest in farm work. For a short time he lived with a relative, a physician, to learn if he had a taste for the medical profession, but it was soon found that this was not the case. His favorite studies were mathematical. With meager instruction and few text-books, he mastered the elements of trigonometry and surveying before he was sixteen.

In 1821, he at last found an opening that might lead to something to his taste. The Erie Canal was then under construction, one section of which was in charge of David Thomas, to whom the young man applied for a position on a surveying party. The only place then vacant happened to be that of camp cook, which Mr. Farnam at once accepted, so as to be on hand when vacancies in the surveying staff might occur. He soon rose to be Assistant Engineer, and was connected with the canal until the fall of 1824, spending the winter months teaching school. It was during this engagement that he contracted a malarial disease that for some time threatened his life. This was finally cured, however, and was practically the only sickness he had until his last few years.

From 1825 until 1850 Mr. Farnam was connected with the canal running from New Haven to Westfield. This work was undertaken in 1822 by the Farmington Canal Company, but actual construction was not begun until three years later. Davis Hurd was Chief Engineer of the company, and he offered the position of Assistant to his relative, Mr. Farnam. It was at once accepted, and in 1827, on the retirement

* Memoir prepared from a biography written by Henry W. Farnam, Esq.

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of Mr. Hurd, was followed by promotion to the position of Chief Engineer. The canal was part of the system of waterways eventually extended from New Haven to Northampton, and controlled by the New Haven and Northampton Canal Company. It was largely built and managed by Mr. Farnam as Chief Engineer and Superintendent, whose experience in this work must have been an anxious one. The balance sheet always showed losses, which were attributed, like those of other New England canals, by its chief financial backer, Joseph E. Sheffield, to two facts—that little passenger travel could be secured, and that the canal never carried on transportation itself, but simply collected tolls, requiring a much larger volume of business to pay dividends than would be needed by a railroad.

By 1845 the company's stockholders were convinced that the canal could not be made to pay, and Mr. Farnam proposed to build a railroad along its line, abandoning the waterway. Mr. Sheffield, then actively engaged in organizing the New York and New Haven Railroad Company, took up the idea, and with his co-operation the railroad was built, reaching Plainville in 1848, and Tariffville and Collinsville in 1850. The New Haven and Hartford Railroad was then in operation. The competition between the two lines was strong, and various deals were made between the roads, which were disappointing to both Mr. Sheffield and his chief engineer. Both withdrew entirely from eastern railroading, but that Mr. Farnam's relations with the New Haven and Northampton Company remained harmonious to their termination is evinced by the following extract from a resolution passed at a meeting of the stockholders in 1850:

"For the uniform fidelity with which Mr. Farnam has performed all the duties devolving upon him; for the unimpeachable integrity with which the many thousand dollars that have passed through his hands have been expended; for the unshaken confidence with which he carried forward these works under very great difficulties; and for the heavy personal responsibility which he often assumed to maintain the works, when otherwise they would have been sacrified, this Company entertain the highest consideration."

Although these twenty-five years were full of arduous works, of long patrols in his buggy along the line of the canal in all sorts of weather, and were rewarded with but a small salary, yet they had one important result, the formation of enduring friendships with men of great business influence and sagacity, particularly James Hillhouse and Mr. Sheffield, and it was in partnership with the latter that Mr. Farnam made his first noteworthy success as a contractor.

In 1850 Chicago was a place of 30 000 inhabitants, built in a straggling fashion on the swamp about the mouth of the nearly stagnant stream known as the Chicago River. Its facilities as a terminus for steamboat navigation were admirable, but otherwise it possessed few attractions. Two years before, the Illinois and Michigan Canal had

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finally been completed from Chicago to La Salle, thus connecting Lake Michigan with the Mississippi Valley. The Michigan Southern Railroad had then reached Hillsdale, 167 miles from Chicago, and a section of the Galena and Chicago Union Railroad ran from Chicago to Elgin. William B. Ogden was then President of the latter road, and at his invitation Mr. Farnam went west, with a view to becoming interested in that undertaking. The possibilities of the country impressed him so favorably, however, that he did not care to commit himself to any one project at that time.

He made another visit a little later in company with Mr. Sheffield, and in their journey through the country were much attracted by the future for a paper railroad between Rock Island and La Salle. When the road was chartered in 1847, its projectors believed that it could not compete with the canal from Chicago to La Salle, but would stand a good chance of securing a profitable traffic from the latter city westward, in competition with the natural waterways. The experience in Connecticut showed, however, that such a project was not the right one, but the firm of Sheffield and Farnam offered to build the road if the charter could be amended to give the right to run through to Chicago.

Before this contract was completed, however, the new firm was given an opportunity to show its capabilities. The Michigan Southern road then stopped at Hillsdale, on account of financial weakness. John B. Jervis was Chief Engineer of the company, and he wanted Mr. Farnam to become Superintendent of the completed part of the line. The latter declined the proffered position, but agreed that his firm would build the road into Chicago, a daring offer in those days. The contract was made, and in March, 1852, the first train from the East entered Chicago over this line. Shortly afterward the Michigan Central built through to the city, which, under the stimulus of these new facilities for transportation, soon began to develop marvelously.

The legislative preliminaries in the Rock Island and La Salle project were meanwhile carried on under the general supervision of the firm. In 1851, the new charter was obtained and the name of the corporation altered to the Chicago and Rock Island Railroad Company, and soon after John B. Jervis was chosen President, and William Jervis Chief Engineer. The surveys were completed in August, and the next month the formal contract with Sheffield and Farnam was closed for building and equipping the entire road, about half the payment to be made in 7 per cent. bonds. Late in February, 1854, the first train ran from Chicago to Rock Island, and in July the road was formally turned over to the company, eighteen months before the time specified in the contract. The firm also built during this time, in company with several other parties, the Peoria and Bureau Valley Railroad, a branch of the Rock Island system. The completion of these contracts

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was celebrated as an event of national importance. Sheffield and Farnam took a party of over a thousand from Chicago to Rock Island, up the Mississippi to Fort Snelling, and back to Chicago. Six days were spent on the excursion, and among the guests were President Fillmore, George Bancroft, Governor Baldwin of Connecticut, Thurlow Weed, Epes Sargent, Charles A. Dana and Samuel Bowles.

Before the completion of the Chicago and Rock Island line, Mr. Farnam had investigated the opportunities for railroads in Iowa, and was convinced that they offered many openings for profitable undertakings. The first thing to be done, however, was to cross the Mississippi, and the Rock Island Bridge Company was formed to do this. The bridge built, the first over the river, was a wooden structure and aroused great opposition. Boats collided with the piers and their owners brought suits for damages, and it was once set on fire. Then the United States Government sued out an injunction against it, but finally the company triumphed over all obstacles and a railroad connection across the river was assured.

The Iowa road was chartered as the Mississippi and Missouri Railroad, and formed part of a single system of which the Rock Island Bridge and Chicago and Rock Island Railroad were the other links. Mr. Sheffield retired from business about this time, and the firm of Farnam and Durant was formed with Dr. Thomas C. Durant. In May, 1855, a contract was taken by the firm for the construction of a road from Davenport to Iowa City, with a branch to Muscatine. A period of commercial depression came on about this time, and during the crisis of 1857, the firm came dangerously near ruin. This was finally avoided, and the road extended to Grinnell, 120 miles from Davenport. During this time Mr. Farnam was acting as President of the Chicago and Rock Island Railroad Company and of a bank, and was also actively engaged in promoting the project of a railway to the Pacific Coast. He finally became one of the incorporators of the Union Pacific Company, but soon found himself out of sympathy with the proposed methods of conducting the enterprise and ceased to have anything to do with it. He continued, however, to take part in railway affairs until June 4, 1863, when he resigned the presidency of the Rock Island Railroad Company, and retired from active work.

Five years of subsequent life were spent in travel and the remainder at his home in New Haven. He was too advanced in years to offer his personal services to the country during the Civil War, but he contributed financial support, and, with his wife, cared for wounded and sick soldiers in Chicago. He made many gifts to Yale College, gave the city of New Haven one of its most beautiful drives, and in many ways used his large wealth for the advantage of others.

His death occurred on October 4th, 1883, and was due to a stroke of paralysis. Mr. Farnam was elected a Fellow of the American Society of Civil Engineers on November 14th, 1872.

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ALFRED KRUPP, F. Am. Soc. C. E.*

DIED JULY 14TH, 1887.

Alfred Krupp's career as an iron master was laid out for him by his father, but the success he achieved was solely his own, and the reward of unremitting labor and bold business enterprise. His father, Friedrich Krupp, was for many years among the most prosperous residents of Essen on the Ruhr, then a little town of about 4 000 people. His great purpose had been to learn the art of making cast steel, which was practiced in England as early as the middle of the last century. During the Napoleonic wars it was difficult in Germany to secure enough of this metal to meet the demands of the manufacturers of fine tools, and many Germans tried to learn the secret. Friedrich Krupp struggled along for years and finally found it. He made steel of good quality, and in an official Prussian report of 1822, it is stated that his product "has been carefully examined by the Bureau of Manufactures and Commerce at Berlin, and has been found, in adaptability and intrinsic excellence, fully equal to the best English steel, in some respects even preferable." But in spite of the good quality of the metal, the demand was very light, and there was not business enough to pay expenses. The pleasant home had to be abandoned, and the family went to live in a small one-story laborer's cottage still standing in the heart of the immense establishment which the genius of the owner's son reared around it.

It was in this humble cottage that Alfred Krupp lived while laying the foundations of the industry that is now known the world over. In 1873 he had an engraving made of the house, copies of which were distributed to the workmen with a note saying, among other things:

"Fifty years ago this laborer's cot was the refuge of my parents. May no workman of ours ever experience the sorrow that then enshrouded us! For twenty-five years the issue was in doubt, an issue which has since then, by degrees, so astonishingly rewarded the privations, the struggles, the confidence and the perseverance of the past. May this example stimulate others in distress, may it increase the respect for small domiciles and the sympathy for the greater cares that often dwell therein."

Shortly before Friedrich Krupp's death, he told his son the secret of making cast steel. In 1826, his widow announced that the works would be continued under the former name and manufacture "cast steel in rods of any thickness, rolled plates and forgings after drawings or models, such as mint dies, shafting, spindles, shear-blades, rolls,

* Prepared chiefly from a biographical sketch by Capt. O. E. Michaelis, U. S. A.

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etc., also tanners's tools." The management of the little plant, which had been a commercial failure under Friedrich Krupp, passed into the hands of his son before the latter was fifteen years of age. With two workmen he labored daily at the forge, and his condition at the time is indicated best by his own words:

"Working hard, often all night long, my food being for the most part potatoes, coffee, bread and butter, but no meat, I felt all the responsibility of a harassed father of a family. For twenty-five years I persevered, until at last, under gradually improving circumstances, I conquered a decent living. My most vivid impression of the distant past is the recollection of the long-continued, ever-threatening danger of ruin, and its avoidance through patience, self-denial and labor."

For a period of about fifteen years, just enough was earned to pay the workmen and living expenses. Under the direction of an uncle, he studied book-keeping and mercantile methods, and gradually extended the market for his steel. In 1832 he had ten workmen, and somewhat later made his first important commercial success, the sale of English patents for a cast-steel roller die. All his spare money was apparently devoted to increasing his plant and for journeys of investigation among English steel works. Early in the forties, he began experimenting with cast-steel guns, and by 1845 the number of employees had risen to 122, although during the agricultural depression and political troubles of 1848, the number fell off for a time. During the latter year Alfred Krupp assumed entire control of the establishment, his brother Friedrich withdrawing, and in a short time the works began to grow in size and importance at a surprising rate. In 1848 the number of men was 72; in 1858, 1 047, and in 1863, 4 185. The most important advances were made between 1850 and 1860. In 1851, Krupp showed at the London Exhibition a cast-steel block weighing over 2 tons, something unheard of in England at the time; in fact, it was so frequently reported that the steel was not of good quality and would fly to pieces like cast iron under the hammer, that a piece had to be cut from the block and forged on the anvil to prove its quality. The next year was marked by the Krupp invention of a method of manufacturing weldless railway ties, which proved very profitable.

In 1853 Krupp married Bertha Eichhoff and left the little cottage, where he had lived during the building up of the works, for an unpretentious two-story house adjoining it. It was here that the present owner of the works, Friedrich Alfred Krupp, was born, and here the family lived until 1864, when they moved to a larger house, also within the limits of the works. Afterward a small country home, a few miles from Essen, was purchased and gradually enlarged. It is worth noticing that for more than forty years the builder of this great establishment lived within its boundaries, most of the time in a house far inferior to that occupied by many of his employees.

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The successful exhibit at London was far surpassed by that at Paris in 1855, where a block weighing 5 000 kilos was shown, which received a gold medal. By this time the manufacture of guns was well advanced, and a 12-pounder shell gun which was among the exhibits was subjected to many trials. This did not lead to an order, however. The first country to order guns was Egypt, in 1857, followed soon by Brunswick and Prussia. About this time he designed his first rifled breech-loaders, which were adopted by Prussia in 1861. In 1868 there was a competitive trial at the Tegel Proving Ground near Berlin between Krupp breech-loaders and Woolwich muzzle-loaders, in which the former proved far superior. From that time on the Krupp ordnance was recognized as among the best, and its service a few years later during the Franco-German war showed that the peace tests were equalled by the hard trials of actual fighting.

The numerous institutions founded by Krupp for the benefit of his employees have been often described and need not be mentioned here. His idea was that "with assured and sufficient earnings, with content and comfort at home, every individual can enjoy the very fact of living," and he endeavored to furnish these conditions as far as possible. His plans have much of the paternalism for which German institutions are noteworthy, but their success is demonstrated by the comparative absence in Essen of labor troubles, anarchistic and even socialistic-democratic agitations, as compared with other important industrial centers in that country.

The success which was achieved by this man was remarkable. In sixty years he won his way from being the boy manager of a nearly insolvent forge, worked by two men besides himself, to the proud position of the heaviest individual tax-payer in the German Empire and the sole owner of a business on which 60 000 people depended for a living. Honors and orders of all sorts were given him by many governments, yet he rarely wore a decoration. Kings, emperors and princes were often among his guests, yet he declined the offer of a title, and preferred to retain the name which his energy had made a guarantee of good material and workmanship.

Early in 1887 his health began to fail. For some years, he had been gradually withdrawing from the business of the works, and leaving the management to a committee of technical, commercial and legal experts of which his son was a member, so when his end drew near there was no hitch in the operations of the establishment, a result he had worked for during a number of years. His strength finally gave out, and on July 14th, 1887, he passed away gently, in the seventy-fifth year of his age.

He was elected a Fellow of the American Society of Civil Engineers on June 14th, 1870.

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WILLIAM C. KINGSLEY, F. Am. Soc. C. E.*

DIED FEBRUARY 21ST, 1885.

William C. Kingsley was born in Fort Covington, Franklin County, N. Y., July 31st, 1833. His father was a farmer, and Mr. Kingsley's boyhood was passed uneventfully upon the farm, where he rendered faithful, painstaking assistance during the summer months. In the winter he attended school, learning readily and showing marked aptitude for mathematical work and great fondness for history.

Before he reached manhood he had acquired such an education as the village school could furnish, and, as he was an omnivorous reader, he had stored away much valuable knowledge gained from works borrowed from neighbors and otherwise obtained, in addition to that acquired at school. Young Kingsley realized very early in life that, attractive as the life of a farmer might be to some, it was not the life for him, and he resolved to leave the parental roof-tree and strike out for himself, a resolution which, once having made, he quickly put into execution.

He started out with the benedictions and blessings of his parents, who had reluctantly consented to his going. He had no definite plan of action, but a fixed purpose to do something which should at least satisfy the yearnings of his soul and enable him to exert to the utmost the powers which he knew were his. He found employment immediately with a railway company which was engaged in constructing a railroad in Westmoreland County, Pa. His duties were clerical, but he brought so much intelligence, fidelity and zeal to bear on their performance as to stamp him at once as a man of no mean ability; but, just as his employers were about to show their recognition of his excellent service, typhoid fever prostrated him, and he lay precariously ill for many weeks. Good habits and previous good health proved their efficient help in assisting him through the terrible illness, but his recovery was slow and the importance of outdoor exercise apparent. Unwilling to remain idle one hour longer than was absolutely necessary, young Kingsley applied for and secured a position as teacher in a school in New Alexandria, without, however, any idea of choosing teaching as a profession. He was singularly successful, acquiring vast influence over his pupils, who recognized in him a master who was a friend as well. His labors in the schoolroom were marked by the same conscientious devotion and industrious, painstaking efforts as had characterized everything he had undertaken from his earliest childhood. During his incumbency as teacher an emeute occurred among the scholars, led by some of the older, more obstreperous pupils. The

*Memoir received through C. C. Martin, M. Am. Soc. C. E.

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skill, judgment and courage with which he met the matter attracted the attention of Col. Snodgrass, who had been accidentally a witness to the fracas. Recognizing immediately the inherent qualities which had enabled the young teacher to so successfully and triumphantly terminate a rather serious difficulty, Col. Snodgrass determined to befriend the young man, and to that end tendered him a position as bookkeeper. Mr. Kingsley accepted the offer, and at once entered upon his duties.

Col. Snodgrass was at this time engaged in building a large canal in Wyoming, Pa. The region thereabout is extremely wild and mountainous, conditions which rendered the work one of great difficulty, which was materially increased by the continual disturbances occurring among the employees, who were, in many cases, as wild as the country about them. Strikes were frequent, and race and clan fights numerous. When young Kingsley arrived, disorganization was complete. He at once, in the absence of his employer, took charge, and in a short time succeeded in restoring order and bringing about a more amicable feeling among the men; gave a new impetus to the work, which had been dragging along for several years, and in a year and a half brought it to a profitable and successful completion.

He remained for eighteen months afterwards with Col. Snodgrass, executing meanwhile for him a contract for the construction of the great tunnel between Altoona and Johnstown. Upon the completion of this work Mr. Kingsley felt himself master of the contracting business, and determined to go west on a tour of observation. He sojourned in Illinois and Wisconsin for some time, and built railroads and other works, always with great success.

Returning to the East, he settled in Brooklyn, then little more than an extensive village. He was immediately recognized as a man of unusual ability and promise, though but twenty-four years of age.

The water-works of Brooklyn were then in process of construction, and a part of the work was sublet to William Kingsley, and the manner in which his part was performed placed him in the front rank among contractors and engineers. About this time he entered into partnership with Col. Abner C. Keeney, a man of great energy and business capacity, and one who proved himself invaluable as a co-laborer and friend. As such he remained until his death. The firm thus formed was awarded the contract for sewering the city of Brooklyn, and more than 65 miles of subways were built under the streets. At the same time, Messrs. Kingsley and Keeney built the Wallabout improvements, and railway lines in Pennsylvania and New York. The Hudson Avenue and the Third Avenue sewers, the stone walls around Central Park in New York, and Washington Park in Brooklyn, were also built by this firm. The greatest of their undertakings as partners was the storage reservoir at Hempstead, N. Y. Many difficulties in excavating and other matters had to be met and overcome in order

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to create the basin, which was built to hold a million gallons of water in reserve. When completed it was flawless, and remains so to-day.

To William C. Kingsley, more than to any other man, are Brooklyn and New York indebted for the great bridge which spans the East River. He conceived the project, and, after consultation with an eminent engineer, who framed the tremendous scheme, set himself to work to overcome monster objections and obstacles of every kind, to effect legislation in favor of the great enterprise. Not once did he falter, though assailed and vilified and hampered in all possible ways. For two years he lived with this thought uppermost in his mind, always actively urging it, and finally securing the necessary legislation authorizing the commencement of the work. A private corporation was first formed, with the two cities as contributors, Brooklyn contributing two-thirds, and New York one-third, to the capital stock, and the incorporators subscribing half a million dollars themselves as an earnest of their faith in the project. Of this sum Messrs. Kingsley and Keeney subscribed over three-fifths. It is not the purpose of this memoir to enter exhaustively into the various stages of the construction of the bridge. Suffice it to record that colossal obstacles and hindrances—the result in some instances of ignorance, in others of jealousy—were constantly being placed in the way as barriers to its successful termination. A less courageous man than William C. Kingsley would have failed in the work, but neither the virulence of his enemies, nor the ignorance of the many who sought to prove its impracticability, served to turn him a hair's breadth from the path he had laid out. The bridge must be built, and he must not and would not rest until it was an actuality. From a private enterprise, the building of the bridge became a public undertaking, largely through the influence of Mr. Kingsley, who foresaw its necessity to insure the success of the undertaking.

He became a member of the Board of Trustees, and of every successive board he continued a member. Through him the services of John A. Roebling were secured as Engineer of the bridge. From that moment the engineering success of the structure was assured.

It is a well-established fact that to the genius of Roebling as an engineer, and to that of Kingsley as a builder, are directly traceable the successful construction of the bridge.

John A. Roebling met his death in the performance of his duty, and his son, Washington A. Roebling, was appointed as a worthy successor of his father, and, though prostrated for months by an illness contracted in the work, directed the construction to the end.

On May 24th, 1883, the bridge was formally presented to the two cities. In that hour, all the difficulties which had beset his path, were forgotten by William C. Kingsley in the supreme vindication which was his, in the enduring monument which was to ever after stand attesting the triumph of his genius and fortitude.

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Having completed his work as builder, Mr. Kingsley tendered his resignation as a Trustee, which was accepted with the deepest sorrow by his coadjutors, who honored and loved him.

Mr. Kingsley was a man of imposing stature and figure, and, wherever he appeared, at once commanded respectful attention. Modest to a degree, he never sought nor toiled for fame.

In politics he was a Democrat, and many a campaign victory could be directly traced to his magnificent skill in organizing party forces. Mr. Kingsley, though importuned again and again to fill very responsible and honorable offices, never did so, but was content to wield his influence in a way best calculated to advance the interests of the country and community.

He was elected a Fellow of the American Society of Civil Engineers on June 6th, 1870.

His death occurred on February 21st, 1885, after a week's illness, with pneumonia and peritonitis. The entire city mourned the loss of a citizen who had served it with all the forces he could command, in a path of unswerving integrity.

FREDERICK W. MERZ, F. Am. Soc. C. E.*

DIED DECEMBER 8TH, 1883.

Frederick W. Merz was born in Germany in 1832, and came to this country about 1850. He was a mechanic in Louisville and opened a shop there for making architectural ironwork and doing general blacksmithing. His early work was on a small scale, and his financial resources were evidently not large, for his first anvil, weighing 140 lbs., he carried on his shoulders from the store where it was purchased to his shop, four or five blocks distant. In the course of time his business grew steadily; by 1870 he had a shop employing about a hundred hands and did a fair share of the architectural ironwork in and near Louisville, Ky. Later on, the business was incorporated, and Mr. Thomas J. Wood was associated with him. He acquired considerable property, and withdrew from his Louisville business four or five years before his death. He went to New York and there lost nearly all his estate, which so preyed on him that he became insane. He was in an asylum in New York for a time, but recovered and went to Louisville. He was attacked a second time, late in 1883, and died at the Central Kentucky Asylum for the Insane, at Lakeland, on December 8th, 1883, of paralysis.

Mr. Merz was elected a Fellow of the American Society of Civil Engineers on May 28th, 1872.

* Memoir prepared from information furnished by W. R. Belknap, F. Am. Soc. C. E.

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McREE SWIFT, F. Am. Soc. C. E.*

DIED APRIL 5TH, 1896.

McRee Swift was born in New York City, April 19th, 1819. He was the son of General Joseph G. Swift, the first graduate of the United States Military Academy and formerly Chief of Engineers, United States Army, and Louisa M. Walker, daughter of James Walker, a rice planter of Wilmington, N. C.

After spending nearly a year at college, he was appointed in 1836 a junior assistant on the surveys of the Long Island Railroad, under General W. G. McNeill, Chief Engineer, and James P. Kirkwood, Past-President Am. Soc. C. E., Resident Engineer. Later in the same year he went to Fort Caswell, N. C., where he pursued professional studies under the direction of his brother, Alexander J. Swift, of the Corps of Engineers, United States Army. The next six years he spent in railway engineering in New England. He was engaged on the surveys and construction of the Boston and Albany (Western) Railroad under Major W. Whistler, Chief Engineer, and Captains William H. Swift and John Childe, Resident Engineers. Subsequently he served as Assistant Engineer under Frederick Harbach, Chief Engineer, on the surveys and construction of the Pittsfield and North Adams Railroad, and the extension of the New Haven and Hartford Railroad to Springfield, Mass.

In 1843 Mr. Swift was appointed to a more responsible position, that of Superintending Engineer of the Wilmington and Raleigh Railroad, extending from the Roanoke River to Wilmington, N. C., a distance of 160 miles. While connected with this company he began to substitute the T-rail for the old plate rail in general use at that time on southern railroads. In 1846 he came North to accept the position of Superintending Engineer of surveys and construction of the Newburgh branch of the New York and Erie Railroad under Major T. S. Brown, Chief Engineer, and served at the same time as commissioner for purchasing land for right of way. In 1849 and 1850 he was Superintending Engineer of the surveys and construction of 70 miles of the New York and Erie Railroad, from a point west of and near Hornellsville, N. Y., to a point below Olean, N. Y. Horatio Allen, Past-President Am. Soc. C. E., was Consulting Engineer for this work. In 1851 he was Chief Engineer of the surveys for the Rochester and Genesee Valley Railroad.

Mr. Swift went to Europe in 1851 with his father, and spent a year in traveling there. On his return he became Chief Engineer of the surveys and construction of the Rochester and Genesee Valley, and

* Memoir prepared from information furnished by his family and from papers on file at the House of the Society.

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the Avon, Geneseo and Mount Morris Railroads. This work engaged his attention until 1856, when he became Engineer and Superintendent of a manufacturing and constructing company, of which he was subsequently elected President.

In 1874 Mr. Swift was elected a member of the Commission on Streets and Sewers of New Brunswick, N. J., and served as President of the Commission for eight years, when he resigned.

Mr. Swift's career was that of an able, upright and successful engineer during the infancy of the profession in this country, but, except to the few professional contemporaries who survive him, he has been best known as a man of affairs and a public-spirited citizen.

His relations, friends, and all who have been associated with him in the many business interests with which he was connected, benefited greatly by his example and his advice, based upon a keen intelligence, generosity and unswerving integrity, never to be forgotten by those who knew him.

The rare cultivation of mind and the manner and presence, which rendered him a typical gentleman of the old school, were combined with business ability of the first order.

All who knew him were benefited by his acquaintance and example, and will remember him as a type of what a Christian gentleman and loyal friend and fellow-citizen should be and can be, and yet consistently achieve success in a professional and business career.

Mr. Swift was the twenty-third member of the American Society of Civil Engineers. His name is among those on the original list of names proposed by the Board of Direction of the American Society of Civil Engineers and Architects, preserved in the Society House, and his membership dated from November 6th, 1852. He became a Fellow of the Society on March 9th, 1870. In 1888 he resigned from active membership, but he retained his interest in the Society to the last, and in a codicil to his will, drawn up a short time before his death, bequeathed the sum of $1 000 to the Society, the income to be devoted to the purchase of rare books and maps for its library and models for its museum. This gift was made in memory of his father, General Joseph G. Swift, who was chief engineer of many undertakings after his resignation from the army, among them the New Orleans and Pontchartrain Railroad in 1829, and the Harlem Railroad in 1832.

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TRANSACTIONS

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American Society of Civil Engineers.

INDEX.

VOLUME XXXVI.

DECEMBER, 1896.

NOTE.—In this Index, Subjects and Writers are given Alphabetically. Titles of Papers are enclosed by quotation marks when indexed alone or under the Author's name. When indexed under the name of a person discussing a paper, the titles are not so enclosed. Titles of Papers are entered with reference to the subject and also under the first significant word.

Digitized by Google

INDEX.

VOLUME XXXVI.

DECEMBER, 1896.

ABBOT, F. V. Single Jetty Harbor Improvement, 132.

ABBOTT, JOB. Memoir of— 538.

ADAMS, ARTHUR L.
"Astoria City Water-Works." (With Discussion.) 1, 62.
Flow of Water in 28 to 42-In. Pipes, 212.

AIR, COMPRESSED.
"A Water Power and Compressed Air Transmission Plant for the North Star Mining Company, Grass Valley, Cal." A. D. Foote. (With Discussion.) 171.

AIR COMPRESSORS.
For North Star Mining Company. A. D. Foote, 180.

ALLARDT, G. F.
Cost of Hydraulic Dredging on the Pacific Coast, 126.
Deepening the Bar of Honolulu Harbor, 125.
Single Jetty Harbor Improvement, 125.

ALLEN, WILLIAM ALBERT. Memoir of— 539.

ALUMINUM.
"Experiments on the Protection of Steel and Aluminum Exposed to Sea Water." (With Discussion.) A. H. Sabin, 483.

ASPHALT.
—for reservoir lining. Arthur L. Adams, 28; Rudolph Hering, 51; E. Sherman Gould, 55.

ASPINWALL, WILLIAM HOWLAND. Memoir of— 598.

"ASTORIA CITY WATER-WORKS." Arthur L. Adams, 1.
Discussion: Rudolph Hering, 50.
Correspondence: Edmund B. Weston, 51; S. Bent Russell, 52; James H. Harlow, 52; E. Sherman Gould, 54; Franklin Riffle, 56; David C. Henny, 58; Arthur L. Adams, 62.

Digitized by Google

BACHE, Alexander Dallas. Memoir of— 522.

BARNES, James. Memoir of— 540.

BERESFORD, Frank. Memoir of— 594.

BLISS, Henry Isaac. Memoir of— 541.

BLUNDEN, Henry D. Memoir of— 542.

BOUSCAREN, G. Suspension Bridges, 462.

BOWLES, Francis T. Protection of Steel and Aluminum, 474.

BREAKWATER.

Effect of Waves on— At Havre, France. Charles Macdonald, 159.
Proper Profile for Resisting Wave Action. Robert Fletcher, 514.

BREITHAUPT, W. H. Suspension Bridges, 437.

BRIDGES.

"Condition of Steel in Bridge Pins." A. C. Cunningham. (With Discussion.) 91.
"The Reconstructiion of Grand River Bridge." W. A. Rogers. (With Discussion.) 341.
"Suspension Bridges.—A Study." Geo. S. Morison. (With Discussion.) 359.

BRIGGS, Robert. Memoir of— 542.

BUCK, L. L. Suspension of Solids in Flowing Water, 325.

CABLES.

—for Suspension Bridges. Geo. S. Morison, 368, 474; F. Collingwood, 428; G. Lindenthal, 447; G. Bouscaren, 462.

CASS, George Washington, Jr. Memoir of— 599.

CHIBAS, E. J. "Construction of a Light Mountain Railroad in the Republic of Colombia." (With Correspondence.) 65, 89.
New Water-Works of Havana, Cuba, 233.

CHRISTIAN, G. L. New Water-Works of Havana, Cuba, 234.

CLARKE, Thomas Curtis.

New Water-Works of Havana, Cuba, 232.
Suspension of Solids in Flowing Water, 325.
Suspension Bridges, 417.

COBB, E. S. "The Problem of the Wheel," 184.

COBB, Robert Linah. Memoir of— 545.

COLBURN, Zerah. Memoir of— 546.

COLLINGWOOD, F. Suspension Bridges, 427.

COMPRESSED AIR.

—for Power Transmission. A. D. Foote, 171.

CONCRETE.

—for Reservoir Lining. Arthur L. Adams, 28; Rudolph Hering, 51; E. Sherman Gould, 223; Emile Low, 235.

Digitized by Google

"CONDITION OF STEEL IN BRIDGE PINS." A. C. Cunningham, 91.
Discussion: William Metcalf, 100.
Correspondence: Geo. T. De Forest, 102; George S. Morison, 102; A. C. Cunningham, 104.

CONNOR, ADDISON. Memoir of— 551.

"CONSTRUCTION OF A LIGHT MOUNTAIN RAILROAD IN the Republic of Colombia." E. J. Chibas, 65.
Correspondence: Robert A. Cummings, 86; E. Sherman Gould, 86; José R. Villalon, 87; E. J. Chibas, 89.

COOPER, THEODORE. "Some General Notes on Ocean Waves and Wave Force." (With Discussion.) 139, 169.
Tides in the Thames, Bristol Channel, Bay of Fundy and Cherbourg, 140.
Suspension Bridges, 428.

COXE, ECKLEY BRINTON. Memoir of— 552.

CROWELL, FOSTER.
Protection of Steel and Aluminum, 498.
Suspension of Solids in Flowing Water, 325.
Suspension Bridges, 464.

CUMMINGS, ROBERT A.
Light Mountain Railroad in Colombia, 86.
Life of Florida Pine and Cedar, 86.

CUNNINGHAM, A. C. "The Condition of Steel in Bridge Pins." (With Discussion.) 91, 104.

DE FOREST, GEORGE T.
Specifications for Steel Bridge Pins, 102.

DILLON, SIDNEY. Memoir of— 603.

DOBLE, WILLIAM. Compressed Air Transmission Plant, 191.

DREDGING.
Cost of Hydraulic— on the Pacific Coast. G. F. Allardt, 126.

DURANT, THOMAS C. Memoir of— 602.

EATON, HORACE LAFAYETTE. Memoir of— 554.

EMONTS, WILLIAM ALEXIS GEORGE. Memoir of— 594.

FARNAM, HENRY. Memoir of— 605.

FIRE-GATE.
—at Astoria, Ore. Arthur L. Adams, 42.

FLAMANT, A. Suspension of Solids in Flowing Water, 327.

FLETCHER, ROBERT. Ocean Waves and Wave Force, 161.
"Proper Profile for Resisting Wave Action." (Abstract.) 514.

Digitized by Google

"FLOW OF WATER IN WROUGHT AND CAST IRON PIPES from 28 to 42 Ins. in Diameter." Isaac W. Smith, 197.
Correspondence: Arthur L. Adams, 212; L. J. Le Conte, 212; Isaac W. Smith, 214.

FOOTE, ARTHUR DE WINT FOOTE. "A Water Power and Compressed Air Transmission Plant for the North Star Mining Company, Grass Valley, Cal." (With Discussion.) 171, 193.

FORMULAS.
For Oscillatory Waves. Theodore Cooper, 144.

GILLISS, JOHN ROBERTS. Memoir of— 555.

GOULD, E. SHERMAN.
Astoria City Water-Works, 54.
Life of Southern Pine, 87.
Light Mountain Railroad in Colombia, 86.
"New Water-Works of Havana, Cuba." (With Discussion.) 217, 237.
Reservoir Lining at Havana, Cuba, 55.
Water Flow in Pipes, 54.

GRANT, WILLIAM HARRISON. Memoir of— 557.

GREINER, J. E. Suspension Bridges, 467.

GRIDLEY, VERNON HILL. Memoir of— 595.

HARBORS.
"Improving the Entrance to a Bar Harbor by a Single Jetty." T. W. Symons. (With Discussion.) 109.
Deepening the Bar of Honolulu. G. F. Allardt, 125.

HARLOW, JAMES H. Astoria City Water-Works, 52.

HASTINGS, L. M. Protection of Steel and Aluminum, 504.

HATFIELD, ROBERT G. Memoir of— 558.

HAUPT, LEWIS M. Single Jetty Harbor Improvement, 133.

HENNY, DAVID C.
Astoria City Water-Works, 58.
Wood-Stave Pipe, Use of— 58.

HERING, RUDOLPH.
Asphalt Lining for Reservoirs, 51.
Astoria City Water-Works, 50.
Ocean Waves and Wave Force, 159.
Water Flow in Pipes, 50.
Wood-Stave Pipe, 51.

HILDENBRAND, W. Suspension Bridges, 454.

HILDRETH, RUSSELL WADSWORTH. Memoir of— 596.

HOBSON, R. P. Protection of Steel and Aluminum, 500.

Digitized by Google

HOOKER, ELON HUNTINGTON. "Suspension of Solids in Flowing Water." (With Discussion.) 239, 340.

HYDRAULICS.

Flow of Water in Pipe. Arthur L. Adams, 26, 212; Rudolph Hering, 50; Edmund B. Weston, 51; E. Sherman Gould, 54; L. J. Le Conte, 212; Isaac W. Smith, 214.

Flow of Water in Wrought and Cast-Iron Pipes From 28 to 42 Ins. in Diameter." Isaac W. Smith. (With Discussion.) 197.

"Some General Notes on Ocean Waves and Wave Force." Theodore Cooper. (With Discussion.) 139.

"Suspension of Solids in Flowing Water." Elon Huntington Hooker. (With Discussion.) 239.

"IMPROVING THE ENTRANCE TO A BAR HARBOR BY A Single Jetty." T. W. Symons, 109.

Discussion: G. F. Allardt, 125; G. H. Mendell, 127.

Correspondence: F. V. Abbot, 132; Lewis M. Haupt, 133; T. W. Symons, 135.

JETTIES.

"Improving the Entrance to a Bar Harbor by a Single Jetty." T. W. Symons. (With Discussion.) 109.

JOINTS, FLEXIBLE.

—at Astoria, Ore. Arthur L. Adams, 42.

KAUFMAN, GUSTAVE. Suspension Bridges, 469.

KINGSLEY, WILLIAM C. Memoir of— 612.

KRUPP, ALFRED. Memoir of— 609.

LE CONTE, L. J.

Flow of Water in 28 to 42-In. Pipes, 212.

New Water Works of Havana, Cuba, 236.

Suspension of Solids in Flowing Water, 338.

LINDENTHAL, GUSTAV. Suspension Bridges, 439.

LOW, EMILE. New Water Works of Havana, Cuba, 234.

MACDONALD, CHARLES.

Effect of Waves on Breakwater at Havre, 159.

Ocean Waves and Wave Force, 159.

MALÉZIEUX, ÉMILE. Memoir of— 524.

MANGANESE.

—in Republic of Colombia. E. J. Chibas, 82.

Limit of— in Steel Pins. Wm. Metcalf, 101.

Effect of— in Steel. A. C. Cunningham, 105.

MAYER, JOSEPH. Suspension Bridges, 420.

MEEM, J. C. Suspension Bridges, 470.

MENDELL, G. H. Single Jetty Harbor Improvement, 127.

Digitized by Google

MERZ, FREDERICK W. Memoir of— 615.

METCALF, WILLIAM.
Limit of Manganese in Steel Pins, 101.
Steel in Bridge Pins, 100.

MORISON, GEO. S.
Steel in Bridge Pins, 102.
Specifications for Steel Bridge Pins, 103.
"Suspension Bridges.—A Study." (With Discussion.) 359, 471.

"NEW WATER-WORKS OF HAVANA, CUBA." E. Sherman Gould, 217.
Discussion: Thomas Curtis Clarke, 232; Edward Wegmann, 232; E. J. Chibas, 233.
Correspondence: G. L. Christian, 234; Emile Low, 234; L. J. Le Conte, 236; E. Sherman Gould, 237.

NICHOLS, NORMAN JAMES. Memoir of— 559.

NORTH STAR MINING COMPANY.
"A Water Power and Compressed Air Transmission Plant for—" A. D. Foote. (With Discussion.) 171.

NOYES, ALBERT FRANKLIN. Memoir of— 560.

PARSONS, WILLIAM BARCLAY. Protection of Steel and Aluminum, 495.

PARTIOT, LEON. Suspension of Solids in Flowing Water, 326.

PHINNEY, HENRY WARD BEECHER. Memoir of— 563.

PINS, BRIDGE.
"Condition of Steel in Bridge Pins." A. C. Cunningham. (With Discussion.) 91.
Tests of— A. C. Cunningham, 96.
Limit of Manganese in— Wm. Metcalf, 101.
Specifications for— Geo. T. De Forest, 102; Geo. S. Morison, 103; A. C. Cunningham, 104, 106.

PIPES.
Flow of water in— Arthur L. Adams, 26; Rudolph Hering, 50; Edmund B. Weston, 51; E. Sherman Gould, 54; Isaac W. Smith, 197.
Tuberculation in— Isaac W. Smith, 211.

PIPE, STEEL.
Frictional Loss in— Arthur L. Adams, 25.

PIPE, WOOD-STAVE.
Life, capacity and advantage of— Arthur L. Adams, 10; Rudolph Hering, 51; David C. Henny, 58.
Maximum pressure for— Arthur L. Adams, 13.
Essential requirements of— Arthur L. Adams, 15.

POPE, WILLARD S. Memoir of— 565.

POST, JAMES CLARENCE. Memoir of— 569.

Digitized by Google

POWER, TRANSMISSION OF.
Efficiencies with Compressed Air and with Electricity. C. G. Young, 192.
Efficiency of Compressed Air. A. D. Foote, 182; E. A. Rix, 186.
"PROBLEM OF THE WHEEL." E. S. Cobb, 184.
"PROPER PROFILE FOR RESISTING WAVE ACTION." (Abstract.) Robert Fletcher, 514.
PROSSER, THOMAS. Memoir of— 564.
RAILROADS.
"Light Mountain Railroad in the Republic of Colombia." E. J. Chibas. (With Discussion.) 65.
"Resurvey of the Williamsport Division of the Philadelphia and Reading Railroad." (Abstract.) George D. Snyder, 509.
RAILS, STEEL.
Manganese in— A. C. Cunningham, 106.
RAYMOND, C. W. Suspension Bridges, 458.
"RECONSTRUCTION OF GRAND RIVER BRIDGE." W. A. Rogers, 341.
Correspondence: Onward Bates, 356; Emile Low, 356.
RESERVOIR.
Construction of— for Astoria Water-Works. Arthur L. Adams, 27
Asphalt lining for— 28; Rudolph Hering, 51; S. Bent Russell, 52.
Palatino – at Havana, Cuba. Lining of— E. Sherman Gould, 55.
Accident to— At Saltville, Va. Emile Low, 234.
"RESURVEY OF THE WILLIAMSPORT DIVISION OF THE Philadelphia and Reading Railroad." (Abstract.) George D. Snyder, 509.
RIFFLE, FRANKLIN. Astoria City Water-Works, 56.
RIX, E. A. "Summary of Tests of the North Star Power and Transmission Plant," 186.
ROBERTS, WILLIAM MILNOR. Memoir of – 531.
ROGERS, W. A. "Reconstruction of Grand River Bridge." (With Discussion.) 341.
ROPES, WIRE.
Tests of— at U. S. Arsenal, Watertown, Mass. Geo. S. Morison, 414; Theodore Cooper, 429; E. Gybbon Spilsbury, 435.
RUDLOFF, HENRY FREDERICK. Memoir of— 570.
RUSSELL, S. BENT. Astoria City Water-Works, 52.
Lining for reservoirs, 52.
SABIN, A. H.
"Experiments on the Protection of Steel and Aluminum Exposed to Sea Water." (With Discussion.) 483, 506.

Digitized by Google

ST. JOHN, ISAAC MUNROE. Memoir of— 571.
SCHUYLER, HOWARD. Memoir of— 572.
SEELY, THOMAS JENNINGS. Memoir of— 574.
SHERRERD, M. R. Protection of Steel and Aluminum, 496.
SHREVE, SAMUEL HENRY. Memoir of— 576.
SICKELS, FREDERICK ELLSWORTH. Memoir of— 577.
SITES, WILMON W. C. Memoir of— 582.
SLIDE RULE. Use in Railway Surveys. G. D. Snyder, 510.
SMITH, ISAAC W. "Flow of Water in Wrought and Cast-Iron Pipes from 28 to 42 Ins. in Diameter." (With Discussion.) 197, 214.
SNYDER, GEORGE D. "A Resurvey of the Williamsport Division of the Philadelphia and Reading Railroad." (Abstract.) 509.
"SOME GENERAL NOTES ON OCEAN WAVES AND WAVE Force." Theodore Cooper, 139.
Discussion: Charles Macdonald, 159.
Correspondence: Rudolph Hering, 159.
SPECIFICATIONS.
—for Steel Bridge Pins. Geo. T. De Forest, 102; Geo. S. Morison, 103; A. C. Cunningham, 104.
SPILSBURY, E. GYBBON. Suspension Bridges, 435.
STAHLBERG, ALBERT JACOB. Memoir of— 597.
STANWOOD, JAMES HUGH. Memoir of— 590.
STARLING, WILLIAM. Suspension of Solids in Flowing Water, 328.
STEEL.
"Condition of— in Bridge Pins." A. C. Cunningham. (With Discussion.) 91.
"Experiments on the Protection of Steel and Aluminum Exposed to Sea Water." (With Discussion.) A. H. Sabin, 483.
Tests of— Pins. A. C. Cunningham, 96.
Limit of Manganese in— Pins. Wm. Metcalf, 101.
Specifications for— Pins. Geo. T. De Forest, 102; Geo. S. Morison, 103; A. C. Cunningham, 104, 106.
Effect of Manganese in— A. C. Cunningham, 105.
STILES, W. M. Protection of Steel and Aluminum, 496.
"SUSPENSION BRIDGES.—A STUDY." George S. Morison, 359.
Discussion: T. C. Clarke, 417; Joseph Mayer, 420; F. Collingwood, 427; Theodore Cooper, 428; E. Gybbon Spilsbury, 435; W. H. Breithaupt, 437.
Correspondence: Gustav Lindenthal, 439; W. Hildenbrand, 454; C. W. Raymond, 458; G. Bouscaren, 462; Foster Crowell, 464; J. E. Greiner, 467; Gustave Kaufman, 469; J. C. Meem, 470; T. Kennard Thomson, 471; George S. Morison, 471.

Digitized by Google

"SUSPENSION OF SOLIDS IN FLOWING WATER." Elon Huntington Hooker, 239.
Discussion: T. C. Clarke, 325; L. L. Buck, 325; Foster Crowell, 325.
Correspondence: Leon Partiot, 326; A. Flamant, 327; William Starling, 328; George Y. Wisner, 336; L. J. Le Conte, 338; Elon Huntington Hooker, 340.

SURVEYS. Grays Harbor, Washington. T. W. Symons, 116.

SWIFT, McRee. Memoir of— 616.

SYMONS, T. W. "Improving the Entrance to a Bar Harbor by a Single Jetty." (With Discussion.) 109, 135.

THOMPSON, John Chambers. Memoir of— 584.

THOMSON, T. Kennard. Suspension Bridges, 471.

TIDES.
Mean Range of— in Harbors of the United States. T. W. Symons, 123.
In the Thames, Bristol Channel, Bay of Fundy and Cherbourg. Theodore Cooper, 140.

TIMBER.
Colombian— E. J. Chibas, 76.
Influence of the moon on— E. J. Chibas, 77; José R. Villalon, 87.
Life of Florida Pine and Cedar. Robert A. Cummings, 86.
Life of Southern Pine. E. Sherman Gould, 87; Emile Low, 356.

TOTTEN, Joseph G. Memoir of— 525.

TRUESDELL, Charles. Memoir of— 585.

TUBERCULATION IN PIPES. Isaac W. Smith, 211.

UPHAM, R. D. Protection of Steel and Aluminum, 496.

VALVES.
Air— for steel water pipes. Arthur L. Adams, 23.

VILLALON, José R.
Light Mountain Railroad in Colombia, 87.
Influence of the Moon on Timber, 87.

WALTON, Louis Roberts. Memoir of— 586.

WATER.
"Flow of— in Wrought and Cast Iron Pipes from 28 to 42 Ins. in Diameter." Isaac W. Smith. (With Discussion.) 197.
"Suspension of Solids in Flowing—" Elon Huntington Hooker. (With Discussion.) 239.
Flow of— in Open Channels. Isaac W. Smith, 206.

"WATER POWER AND COMPRESSED AIR TRANSMISSION Plant for the North Star Mining Company, Grass Valley, Cal." Arthur De Wint Foote, 171.
Discussion: S. S. Wheeler, 191; William Doble, 191.

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WATER-WORKS.
"Astoria City—" Arthur L. Adams. (With Discussion.) 1.
"New— of Havana, Cuba." E. Sherman Gould. (With Discussion.) 217.
WAVES.
"Proper Profile for Resisting Wave Action." (Abstract.) Robert Fletcher, 514.
"Some General Notes on Ocean Waves and Wave Force." Theodore Cooper. (With Discussion.) 139.
WEGMANN, EDWARD. New Water-Works of Havana, Cuba, 232.
WESTON, EDMUND B. Astoria City Water-Works, 51.
WHEELER, ORLANDO BELINA. Memoir of— 587.
WHEELER, S. S. Compressed Air Transmission Plant, 191.
WHEELS, WATER
"The Problem of the Wheel." E. S. Cobb, 184.
For North Star Mining Company. A. D. Foote, 171.
WHIPPLE, SQUIRE. Memoir of— 527.
WILSON, JOHN ALLSTON. Memoir of— 588.
WISNER, GEORGE Y. Suspension of Solids in Flowing Water, 336.
WOOD, CHARLES. Memoir of— 591.
YOUNG, C. G. Compressed Air Transmission Plant, 192.

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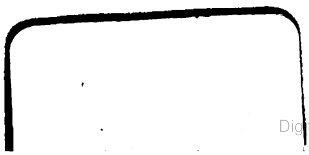

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